

# **APPROPRIATE TECHNOLOGY OPTIONS FOR MANAGING DRAINAGE FLOWS FROM LOW COST PERI URBAN SETTLEMENTS IN SOUTH AFRICA**

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A research report submitted to the Faculty of Engineering and the Built Environment,  
University of Witwatersrand, Johannesburg, in partial fulfillment of the requirements  
for the degree of Masters of Science in Engineering.

Johannesburg, 2004.

## **DECLARATION**

I declare that this research report is my own, unaided work. It is being submitted for the degree of Master of Science in the University of the University Witwatersrand, Johannesburg. It has not been submitted before any degree or examination in any other University.

\_\_\_\_\_  
(Signature of candidate)

\_\_\_\_\_ day of \_\_\_\_\_ (year)\_\_\_\_\_

## ABSTRACT

Inadequate drainage in dense peri-urban settlements in South Africa is a significant problem endangering the public and environmental health and of particular concern are the downstream watercourses, which are a source of drinking water supply, a scarce resource. The objective of this research was to identify appropriate solutions within the limited scope of technical and financial feasibility with reference to Alexandra west bank as a case study area.

The findings show that three physical site conditions hamper the application of onsite drainage approaches in Alexandra west bank Township, the case study area. They are:

- Congestion, due to haphazard development patterns,
- High drainage flow generation resulting from high population densities and the predominantly impermeable surface area due to intensive site development, and
- Poorly draining soils

Congestion, high densities and intensive site development are characteristics common to low-income settlements in South Africa, and they result in lack of space availability for storage facilities, and interference with nature's ability to retard, retain and infiltrate significant quantities of the storm runoff flows. Poor soil drainage capabilities, which is more specific to study area would result in a slow rate of exfiltration of drainage flows that would in turn cause ponding and the associated health hazards.

Estimates of drainage flows generated from the study area as determined from field observations, flow measurements and computer simulation techniques indicate that if the minimum rate of production of just the wastewater component of the drainage flows is taken, which is approximately 37m<sup>3</sup>/ha/day, it exceeds the rate recommended for safe onsite management of drainage flows by almost four times.

Three off-site drainage system arrangements were compared on the basis of the cost of outfall pipe drains sized according to conservative design procedures, and it was found that the combined sullage and storm water drains with separate sewage (black water) drainage system arrangement is more economical than the commonly practiced approach of separate storm and combined sewage and sullage drainage system arrangement.

## **ACKNOWLEDGEMENTS**

I take the opportunity to extend my gratitude to the following people and organizations for their support and guidance in various ways:

GOD the father of my LORD Jesus Christ for HIS gift of eternal life, the unconditional love that builds me up and the assurance of HIS word that I can do all things through Christ who strengthens me. You are faithful

My family who are a great source of support and dependability, my beloved parents who through sacrifice and patience committed themselves to a better life for me, and my sisters and brothers who have continued in the same spirit. God richly bless and reward you in this life.

Professor D. Stephenson for the opportunity to work on this research study, and his guidance and helpful insight through the entire research process.

The Water Research Commission, for facilitating this work, and the steering committee members in the persons of Mr. Jay Bhagwan, Mr. BW de Swardt, Mr. L. Naude and Ms. L A Boyd for providing additional sources of information.

The management and staff of Johannesburg Water particularly Mr. Tony Cross, for allowing me access to information on their activities in Alexandra west bank the study area.

Finally thanks to my friends and extended family, Angelita, Ariyo, and Thomas for praying me through, the Mubangizi's, Richard, Cobb, Barbara and family for going out of your way to help, Daniel, Dr. Lebetse, Dr. Kyamulesire and family for being there for me.

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## **CHAPTER ONE: INTRODUCTION**

### **1.1 GENERAL BACKGROUND**

The 90's in South Africa witnessed the rapid establishment and proliferation of dense informal settlements at a rate that was not matched with the provision of adequate or appropriate services for water supply, sanitation and solid waste management. In some of these settlements (e.g. Alexandra Township) burgeoning populations have over loaded the existing waterborne sewerage infrastructure and the result is sewage overflowing onto the streets from burst pipes. The municipal authority's temporary intervention efforts in the way of chemical toilets have not been effective since these facilities do not provide for 'grey' water disposal. The situation is such that the paths of sewage, sullage and storm water are often merged as during the rainy season the sewage, and grey water flowing on the streets is washed into surface water drains, together with the solid waste littering the streets and ends up in the watercourses. The result is rapidly deteriorating public health and environmental conditions.

In Latin America and South Asia, where various countries are faced with similar problems in the peri-urban areas, (rapid urbanization being a worldwide trend with developing countries being the worst hit), a range of techniques have been employed in drainage of these areas with relative success. These techniques support the control of drainage flows by way of on-site management using direct reuse, storage and/or infiltration methods, as alternatives to the conventional off-site management approaches that have previously been practiced. They have the effect of attenuating peak discharge flows or reducing runoff volumes in the case of storm water while providing some level of treatment for all three waste streams mentioned.

In this research, an assessment of the suitability of the available technologies for the control of drainage flows from Alexandra west bank is made with a view to determining the most appropriate for application in similar peri-urban settlements in South Africa.

### **1.2 PROBLEM STATEMENT**

Rapid urbanization and population growths, coupled with the backlog in provision of drainage infrastructure in peri-urban areas in South Africa has led to the deterioration in the sanitary, aesthetic and environmental conditions of low cost townships in peri-urban as the flow paths of sewage, "grey" water and storm water are often merged and flow down natural

or artificial drainage paths leading to natural watercourses. The haphazard manner of development characteristic of peri urban areas further exacerbates the problem creating difficulties in accessibility for provision of services.

### **1.3 OVERALL AIM**

The overall aim of this research has been to identify the most appropriate technology options for drainage of low cost peri-urban townships with reference to Alexandra west bank as a case study example.

#### **1.3.1 Specific objectives**

The following specific objectives:

- Establish the principles governing drainage of the three major waste streams of concern in low-income peri-urban settlements.
- Identify the available technical options and the factors that impact on the choice of option(s).
- Determine the quantity of flows for each of the three waste streams from Alexandra west bank the study area.
- Establish the cost implications of the feasible technology options for comparison purposes.

### **1.4 RESEARCH METHODS**

The methodology adopted in achieving the objectives outlined included:

- Review of existing literature through Internet searches, desk studies on common characteristics of peri-urban settlements and how they impact on choice of technology, available technology options, methods of determining quantity of individual waste stream flows.
- Hydrological simulation of the study area watershed using computer software program.
- Measurement of sewage flow depths and hence sewage flow rates generated by Alexandra west bank dwellers.
- Field observations of the water use and wastewater disposal practices of the peri-urban settlement dwellers of the Alexandra west bank.

## **CHAPTER TWO: STUDY AREA**

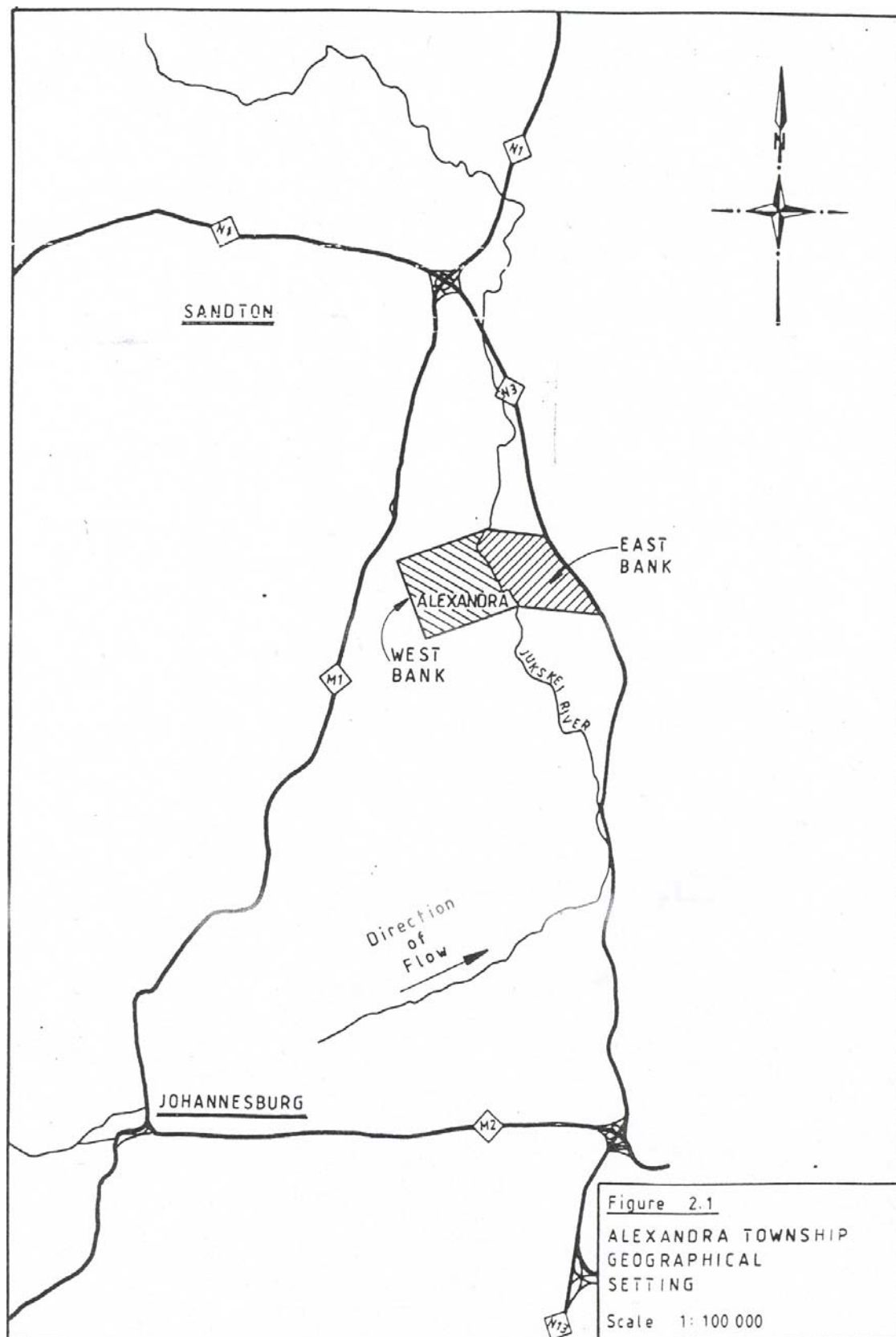
### **2.1 GENERAL DESCRIPTION**

Alexandra west bank, also known as Alexandra Proper is the study area for this research. It is part of Greater Alexandra, situated to the North East of central Johannesburg on a piece of land that was originally a farm belonging to a Mr. Papenfuss. It is bordered by the Jukskei River in the East (which also serves as a separation between the east and west banks of Greater Alexandra), Wynberg and the M1 motorway in the west, in the south by Kevin and Lombardy and north by Malboro gardens (see figure 2.1). (Alexandra Renewal Project (ARP), 2000)

### **2.2 HISTORICAL BACKGROUND**

Development in Alexandra began in 1905, when the original owner sought to establish a portion of his farm as a white residential township. There were no takers at the time because of the considerable distance from the center of Johannesburg, which was barely developed beyond May Fair and Parktown. It was proclaimed a 'native township', in 1912 after the owner resubmitted it, offering land to coloreds and blacks on a freehold basis, prior to the 1913 Land Act, (Wimberley, 1992; ARP, 2000). This made it an attraction to immigrants seeking work and the years 1945 – 1948 brought a large influx of blacks in. The township lacked proper management and resources, and so the area had not been serviced with basic infrastructure, while uncontrolled population growth, unregulated land use development and subsequently congestion continued unchecked. (ARP, 2000)

In the 1950's and 1960's the government sought to reverse this trend and upgrade Alexandra through the construction of single-sex hostels and the forced relocation of the unemployed to Tembisa and Soweto, (ARP, 2000). Local resistance resulted in reversal of the resettlement policy and instead, plans to redevelop Alexandra into a 'model township', which offered accommodation to the black middle class under a 99-year leasehold were drawn up. These plans also fell through, as the residents that were unlikely to benefit from them did not support them, in addition the sheer volumes of the residents to be temporarily moved until new structures become available made them impossible to execute.





Attempts were made to improve management and governance of Alexandra during the 1970s and 1980s. Until that time it had no formal municipality and was being managed by a health committee that wasn't permitted to raise funds for management, (ARP, 2000). In 1983, the Alexandra Town Council was established, which was later incorporated into the Eastern Metropolitan Local Council, (EMLC) of the affluent Sandton area during the 1994 political transition in South Africa (Leitch, 2002). In the build up to the December 2000 local government elections, metropolitan councils were established and thus the EMLC was disbanded and incorporated into the City of Johannesburg metropolitan council, under which Alexandra now falls administratively. (Leitch, 2002)

### **2.3 LAND USE AND DEMOGRAPHICS**

There is no realistic population estimate for Alexandra west bank, however, estimates using aerial photography put it at about 370,000 inhabitants. Other widely varying estimates that take into consideration the dynamic nature of the population have put the figure at between 180,000 – 750,000, (GJMC, 2000). The estimate of 370,000 people occupying approximately 350 hectares of land is used in this report, and this indicates that densities are over 1000 people/ha in the flood plains of the Jukskei River.

The majority of 'Alex' residents are unemployed (the official figure is 60%), despite it being a haven for immigrants seeking employment. About 15% are categorized as indigents, while the working population fall under the low-income bracket i.e. unskilled workers since the overall level of formal learning in 'Alex' is low. The low earnings of the residents are portrayed in their monthly expenditure that is mainly focused on basic necessities particularly food, (ARP, 2000). The findings of a questionnaire survey administered in August 2003 to establish the income levels, water use and wastewater disposal characteristics of the residents in the area are similar. (See appendix B.2)

Alexandra west bank though a previously planned and serviced settlement, is characterized by a haphazard land use pattern that reveals a mixed formal and informal nature both in dwellings and services, (Pegram and Palmer, 1999). It is predominantly residential, with the residential component comprising formal detached housing (about 4000), 'backyard' shacks (about 20,000), hostels and flats. Other land uses include: businesses, such as spaza shops and shebeens and community facilities such as schools, clinics, and churches, (ARP, 2000; GJMC, 2000). There is a shortage of open space primarily because of the encroachment of

informal housing onto public spaces such as pavements, schools and the natural open space system being along the tributaries of the Jukskei River. (ARP, 2000)

## 2.4 SOILS AND GEOLOGY

The geology in Alexandra was investigated by Partridge, De Villiers and Associates, Consulting Engineering Geologists in 1980 and their findings indicate that the area is underlain by highly weathered and decomposed rocks of the Archaen granite, forming the Johannesburg/Pretoria dome (Wimberley, 1992). Outcrops of granite occur in places, especially in the riverbed of the Jukskei River. The granite is, in its fresh state, a medium to coarse textured pink or grey rock.

The most recent geo-technical investigations were carried out in 2002 and 2003 by Moore Spence Jones (Pty) Ltd, the consulting engineers on two construction projects, *The Extension Phase II to RCA Block 102, Alexandra*, and *Proposed Pan African Taxi Terminal and Retail Section in Wynberg*. They were done in fulfillment of the requirements of existing council regulations that stipulate that prior to the approval of building plans, geo-technical investigations are carried out on the proposed site, (Moore Spence Jones, 2002&2003). The sites of these investigations together with sites of recent investigations in the area by Partridge Maude, and Associates are included in appendix E.

The investigations at the site locations for these projects reveal the following about the soil profile:

- A rubble/refuse fill ranging in thickness from 0.3m – 0.9m, with an average thickness of 0.52m. It consists of rubble, bricks, masonry and household refuse in a silty, sandy matrix. Partridge, De Villiers and Associates defined this section of the soil layer as various transported soils as well as unconsolidated fill material. (Wimberley, 1992; Moore Spence Jones, 2002 & 2003)
- Residual granite was encountered to the full depth of most of the test pits (which ranged from 7 – 10m) consists of a loose to medium dense, silty sand. Partridge, De Villiers and Associates encountered this residual soil layer between 0.5 – 6.0 metres.

The geo-technical investigation was carried out in two phases, the first one being at the beginning of the rainy season, at which time ground water seepage was noted at depths ranging from 2.8 – 9m, however in the next phase though no ground water or seepage was encountered although, the soils were recorded as slightly moist to moist, (Moore Spence

Jones, 2002 & 2003). Partridge, De Villiers, and Associates observed that, the position of the bedrock relatively close to the surface gives rise to a perched water table, (Wimberley, 1992).

## **2.5 EXISTING DRAINAGE SITUATION**

Alexandra west bank was a previously planned and serviced settlement (Pegram and Palmer, 1999) and there exists therefore sewerage, storm drainage infrastructure and other services.

### **2.5.1 Sullage and excreta disposal**

The waterborne sewerage infrastructure like other services was originally meant to serve 70,000 people. It comprised of a reticulation system serving the original formal houses that terminated into three pump stations located on the west bank of the Jukskei River. The sewage was then pumped to the Bruma Outfall sewer, (ARP, 2000).

The occupants of shacks that have been developed in the backyards of these formal stands have connected to this system illegally, and because of the overload, there are frequent sewage blockages and surcharges. Congestion makes access for maintenance impossible, as some of the shacks are built over sewer lines and manholes and would have to be demolished. (GJMC, 2000)

The residents that settled and built on the riverbanks and tributaries are not connected to the formal waterborne sewerage system, and have been temporarily provided with chemical toilets (1 per 7 persons), (ARP, 2000). These are a form of bucket latrine system with high operation and maintenance requirements that exploits chemical decomposition of waste. (GJMC, 2000) These banks of chemical toilets have been placed on the periphery of the informal clusters, because of space constraints within the areas and because of the threat of crime, dwellers do not venture out in the night to use the facilities. Overnight containerization of wastewater is therefore practiced and the wastewater disposed of the next day. (GJMC, 2000)

The chemical toilet sanitation technology doesn't allow for sullage disposal. This together with the frequent failure of the existing sewerage system has resulted in sullage and sewage flowing onto the narrow streets and alleys creating very poor sanitary conditions. During the summer rains these wastewaters are washed into the underground storm water drains that discharge into the Jukskei River, and thus it is now transformed into an open sewer.

In a bid to overcome the general overloading of the sewerage system, interceptor sewers have been installed, and they connect to an outfall sewer carrying sewage off the site to the bulk sewerage system. In addition, the flow from the three storm water drainage culverts has been diverted to discharge into the sewer system, thus only the overflow resulting from high intensity rainfall events is discharged in the Jukskei River.

### **2.5.2 Storm drainage**

Alexandra is divided into three major sub catchments identified as the *north*, *middle* and *south* catchments. Each sub catchment is served by a minor and major storm water drainage system, (Stephenson, 2002). The minor system consists of three underground rectangular culvert drains laid in an easterly direction with their outlets releasing into the Jukskei River. The major storms are channeled via the overland escape system, which is along the Jukskei River tributaries, (ARP, 2000). The entire catchment is relatively flat with an average gradient of approximately 0.04. (Stephenson, 2002)

The most recent inspection done along the route of each of the three culverts was carried out by Stephenson and Associates for the Alexandra Renewal Project, (ARP), and their findings on the structural condition and capacity of the drains and the inlets are detailed below:

- The culverts are designed to operate up to at least a 1 in 10 year flood (provided litter can be excluded from them) any excess will overflow on the surface. Water is transferred into them via a series of surface inlets that divert water from the road surface as well as over land flow.
- The drains do not appear to have severe structural damage, although there are leaks in the walls. The major damage occurs outside the drains where there appears to have been soil subsidence and possible washing away of soil besides and under laterally incoming drains.
- Blockages by litter deposited on the top of grid and side inlets, in addition to broken side slabs have greatly reduced the capacity of the drains.

The land around and on top of the culverts is very densely populated with shacks making accessibility for maintenance difficult and blocking the natural pathway for storm water, increasing the effect of the floodwaters. The residents in close proximity to the culverts are thus exposed to: flooding from the culvert overflowing, drowning (if they fall into the culverts as many grid inlet covers are missing and in the event that a section of the culvert collapses), as well as disease since the water flowing in the culvert is dirty. (Stephenson, 2002)

## **2.6 FUTURE DEVELOPMENT PLANS**

This discussion on future development plans shall focus on de-densification, housing, and water supply service levels because they have a bearing on the nature of recommendations that can be made regarding drainage of the settlement.

### **2.6.1 De-densification**

In February 2001, the President of the Republic of South Africa announced plans of the National government to rejuvenate Alexandra Township and R 1.3 billion was allocated to this project. Following the announcement of this initiative multi-disciplinary professional teams were set up to carry out preliminary studies and to guide the renewal development process. De-densification has been recognized as necessary and an optimal population of 300,000 people is being targeted, which will bring the population density down to approximately 857 persons per hectare. Up to 7,000 households have already been relocated from the banks of the Jukskei River to the Diepsloot area, and to other sites where residents are being temporarily housed in warehouses in the neighboring industrial areas. Relocation has been guided by the aim/objective of improving access to shacks and tributaries for maintenance and being able to provide basic service improvements *not* the wholesale clearance of areas.

The other targeted households (approximately 13,000) are the informal settlements along the tributaries of the Jukskei. Relocation will be to the new development areas that have been identified through the ARP, including Frankenweld, Westlake, Rietfontein, Islamic trust area and Malboro South.

### **2.6.2 Housing**

The realization that large plots/stands actually invite the fortunate beneficiary to invest in the construction of unplanned backyard shack to provide rental income has made the case for creating 'special development zones' which permit relaxation of long established planning and engineering standards in the interest of trying and testing new standards and modes of development. (GJMC, 2000)

The housing arrangements proposed for the Alexandra west bank mirror these new standards and include: (GJMC, 2000)

- Formally legalizing backyard shacks, and then working with landlords to improve their configuration and conditions in order to achieve more efficient and healthy layouts ensuring access to services.

(N.B. It has been established that 60% of backyard shacks are in fairly good condition and thus with damp proof course installed, and their ventilation and insulation improved they will be to required standards). (Morkel, 2004)

- The housing stock that can't be upgraded will be demolished and new units built, the property boundaries will be re-aligned into small narrow stands configured for row housing and/or selective adoption of two-story walk up apartment units.

(N.B. The target is to have 65% of the gross stand area developed, however, this is not a rigid target, and is subject to prevailing site conditions) (Morkel, 2004). A maximum of 80% of stand area developed is acceptable.

- Designing schemes with choices for beneficiaries including a variety of plot sizes some of which include some planned backyard rental units seeing that rental income is socially desirable and should not be discouraged. The prices charged would be based on the value of a unit.

The implementation of the above proposals awaits the promulgation of a new legal framework, however in the meantime the ARP is in the process of resolving ownership issues by way of financially compensating claimants and transferring property to rightful owners (i.e. those with strongest ownership claim). Original owners will be required to register claim with a housing transfer bureau that will be established in July 2004 and it will be responsible for award of ownership. (Morkel, 2004)

With these arrangements, high densities are not a problem, and infrastructure would be designed accordingly. These arrangements also ensure more affordable housing and infrastructure provision, and more efficient land use. The environmental and health problems currently being experienced would also be eliminated. (GJMC, 2000)

### **2.6.3 Water supply service levels**

Alexandra's water reticulation system is said to be in good condition having only been replaced in the late 1980's. The upgrading and rehabilitation works that are currently being undertaken are: (DeVallier, 2004)

- Repair and/or replacement of pipes with leakages within the reticulation system
- Removal of all illegal connections
- Construction of proper valve and hydrant chambers
- Reducing pressures and ensuring that the five different pressure zones are discrete.
- Inspection of the 4,556 ablution blocks comprising of a toilet, shower (with tap) and laundry trough, which were installed to serve the informal dwellings erected in the

backyards of the original formal houses, for leakages and carrying out other general repairs.

It is the goal of the ARP: (Morkel, 2004)

- To increase the number of ablution blocks so that for the people living in the old but upgraded backyard shacks 5-6 people share an ablution block i.e. shower, laundry trough and toilet block.
- Individual house connections will be provided for each of the new units constructed in the instances where demolition and not upgrading of backyard shacks is carried out.
- The original formal houses, which already have individual connections, are inspected leakages repaired, and meters installed.

## **CHAPTER THREE: LITERATURE REVIEW**

### **3.1 INTRODUCTION**

In recent years, South Africa has experienced a massive increase in urbanization a large portion of which takes the form of high density, informal settlements that develop around existing metropolitan areas. (Schoeman, Mackay & Stephenson, 2001). South Africa is not alone in this as most developing countries face similar problems (Maksimovic, 2000). These settlements have developed outside of government control and do not follow strictly formal and traditional urban planning and development processes. These settlements are therefore characterized by illegal or uncertain land tenure, minimal or no infrastructure, and lack of recognition by formal governments. (Hogrewe, Joyce & Eduardo, 1993)

The informal status of these settlements and their rapid growth, which by far outstrips the capacity of local governments, has resulted in low levels of sanitation service among other problems, (Hogrewe, Joyce & Eduardo, 1993). The unsanitary conditions arise from a lack of physical infrastructure to remove wastewater generated by settlers, which then flows by gravity through the natural drainage of the landscape ending up in watercourses. During rainfall events, storm water then mixes with wastewater and polluted water is spread of a wider area than the drains. The situation is such that environmental conditions of the area are degraded, and water borne diseases are endemic in these communities, (Maksimovic, 2000). The peri-urban settlements have therefore become a focus as local planners and government seek to address peri urban sanitation needs.

This chapter presents the findings of a review of current literature on both historical and newly emerging trends for managing drainage flows with emphasis being placed on highlighting the technology options available and the procedures that should be followed in arriving at the appropriate technical solution(s) in controlling the quantities of these drainage flows from peri urban settlements.

### **3.2 GENERAL PHILOSOPHY**

Traditionally, drainage has involved collection of storm water, domestic and industrial wastewater flows through buried pipe drain systems or open ditches for disposal to the nearest stream or other water body such as lake, wetland or coastal water (Ho, 2000). This was with the aim of solving local flooding problems by transferring large amounts of runoff



flows downstream of any built up area as fast as possible in the event of a storm while avoiding the health problems caused by wastewater within the community in the case of domestic and industrial wastewater. The storm drain systems discharged directly into the receiving water environment while the wastewater drainage systems were directed to treatment plants prior to final discharge. This approach to drainage hasn't found wide scale application outside of cities of developed countries because of the huge financial investment required for the installation of the service infrastructure and the high skills level required for the operation and maintenance.

The period prior to and during the 1980's highlighted the effects of urbanization on the quantities and quality of runoff generated on the receiving environment, and the inadequate sanitation coverage in the rural and peri-urban areas of developing countries prompting research into alternative drainage solutions that were sustainable in regards to both the economy and environment of beneficiary communities. The alternative drainage philosophy entails:

- The adoption of measures that control flooding and pollution as near to the source as possible by way of emulating natural catchment conditions in the drainage of storm runoff flows so as to maintain peak discharge rates and quantities to levels before the development of site. (Maksimovic, 2000)
- Exploiting the resource potential of wastewaters generated at the household level by way of separation of waste streams for reuse. (Ho, 2000)

The drainage technology options developed and currently in use, which, are based on these principles commonly involve on-site management, which is in direct contrast to the traditional technology options.

In section 3.3 below, drainage technology options that fall under conventional and alternative approaches are described and their role in controlling flows of each of the three waste streams of concern in dense low-income townships in the peri-urban areas of South Africa.

### **3.3 DRAINAGE TECHNOLOGY OPTIONS**

Drainage involves the removal of storm water runoff and polluted water from built up areas in a controlled and hygienic manner in order to minimize public health hazards, inconvenience to residents, and the deterioration of other infrastructure. (Phalafala, 2002; Cotton and Tayler, 2000)

### **3.3.1 Storm water drainage**

Storm water is the water originating from local rainfall occurring within the catchment boundary that isn't intercepted, infiltrated or temporarily stored in natural depressions. It is often the largest stream from built up areas, albeit irregular as dependant on the occurrence of a storm event.

#### **A. Traditional/conventional technical measures in storm runoff control**

Conventionally, storm runoff control has aimed at rapid removal of surface water using a network of separate or combined pipe systems for disposal to the nearest stream or other water body. The aim was to protect developed areas against health hazards, damage to property and interruptions to communications by the accumulation of surface waters. (Barnes et al, 1981)

#### **The combined system**

This system of drainage developed haphazardly and involved the culverting of natural storm drainage paths and later connection of house drains into these drains creating a single pipe system for foul and storm runoff (Phalafala, 2002). With this system, since only one pipe network is required, its capacity is a little more than that of a storm sewer system, since the peak rate of flow of sanitary wastewater is much less. There were problems noted with this system particularly in regard to ensuring a self-cleansing velocity for dry weather flows and the diversion of untreated polluted waters directly into river streams during high intensity rainfall. Recent trends have therefore been aimed towards the development of separate systems for polluted and storm runoff, giving rise to the separate system described below. (Maksimovic, 1998)

#### **The separate storm drainage system**

This system consists of open channels or underground pipes with a multitude of inlets or open ditches designed to carry the runoff flows generated by minor but frequent storm events (i.e. 1 in 2yr, 1 in 5yr, 1 in 10 yr) while the major storm events overflow to roadways. (CSIR, 2000; Stephenson, 2001)

The open channel drains are relatively simple to construct and maintain, they however, have large space requirements and access must be restricted or they pose a hazard to road users especially when very wide, deep or pass along a busy public road or street. They are also susceptible to abuse by residents who throw rubbish into them, reducing their capacity and eventually blocking the flow. The underground drains on the other hand are an advantage where surface space is a premium however, access for maintenance is a problem (Cotton and Tayler, 2000).

## B. Alternative storm runoff control options

The alternative storm drainage solutions evolved upon realization that with urbanization, in addition to the effects of flooding, the impacts from increased pollutant export on receiving waters require consideration, (Schueler, 1987; Maksimovic, 2000). They are variously called sustainable urban drainage systems (SUDS), best management practices (BMP) (Maksimovic, 2000) and they find application within the social, institutional and technical contexts. The technical options function in the following ways:

- Reduction of flows entering the drainage system by diverting a significant fraction of storm runoff volume in the soil.
- Attenuating flows within the drainage system by providing opportunities for the storage and slow release of flood waters and thereby delaying the time for peak flow rate and magnitude.

These management objectives are achieved through the use of infiltration and storage devices strategically located along the drainage path. They are described here below:

### **Infiltration BMP's**

Infiltration systems are generally effective in reduction of storm runoff volumes by detaining the runoff, and allowing it to infiltrate into the soil, (Nsibirwa, 2003). The infiltration devices can be located at the ground surface, in which case they are *infiltration basins* or located below the ground surface and are referred to as *soakaways, infiltration reservoirs and trenches* into which runoff is directed. (Maksimovic, 2000)

*Infiltration basins* are impoundments in which storm water is captured and slowly infiltrates into the surrounding soil through both the bottom and sidewalls. They are said to closely reproduce natural, pre-development hydrologic conditions, since they can be adapted to provide both a reduction in runoff volume and a shaving of peak discharge rates. Several modifications to the original design have been developed to accommodate the short and long term limitations of site conditions, concentrated flows and sediment loads generated from larger watersheds. (Maksimovic, 2000; Schueler, 1987)

This BMP is feasible where soils are permeable and the water table and bedrock are situated well below the soil surface. They can serve relatively large drainage areas of residential or commercial sites (i.e. up to 50 acres) however, some designs are limited to between 5 – 20 acres only. (Schueler, 1987)

*Infiltration trenches* are shallow excavated ditches that have been in-filled with stone to provide underground storage (Maksimovic, 2000). This infiltration BMP is seldom practical on sites larger than 5-10 acres, and also requires permeable soils as well as water table and

bedrock to be situated well below the bottom of trench. They can fit into margins; perimeters; and other unutilized areas of a site thus suitable for sites where space is limited. Various designs exist that can provide peak discharge, volume and/or water quality control depending on the intentions of designer. (Schueler, 1987)

They are not intended to trap coarse sediments but effectively remove both soluble and particulate pollutants. They function by storing water temporarily and allowing water to infiltrate into the ground. Through them significant ground water recharge, low flow augmentation and localized stream bank erosion control can be achieved. (Schueler, 1987)

### **Storage BMP's**

The ground surface storage reservoirs are of three types, namely:

***Detention basins***, which store water temporarily and, normally have dry beds, they permit settlement of coarse silts. They are normally sited in open space areas like parks and recreational grounds and are often placed on-stream with controlled outlets, or they can be placed off channel too. They are helpful in attenuating storm runoff flows as the discharge rate from pond can be controlled, (Stephenson, 1981). The off channel ponds provide a more effective peak flow attenuation than on channel ponds for a given storage volume as they start to fill up once a certain discharge is exceeded within the drainage system.

In addition to peak flow attenuation, detention basins provide moderate to high removal rates of settle able pollutants but this is dependant on pond depth. Extremely shallow ponds are prone to re-suspension of settled sediments caused by wind action, while extremely deep ponds are prone to thermal stratification, which can lead to anoxic and non ideal settling conditions. (Nsibirwa, 2003)

***Retention ponds***, also known as wet ponds permanently divert part of or all runoff expected from a particular drainage area, depending on the design. The flows diverted seep through the soil or they are evaporated i.e. they don't return to drainage system. (Stephenson, 1981) This storage BMP type is referred to as multi-purpose because: (Schueler, 1987)

- It can receive storm runoff from large drainage areas (i.e. greater than 20 acres)
- It improves on storm water quality because it provides a longer hydraulic retention time and is inhabited by aquatic vegetation, which facilitate nutrient uptake in addition to allowing for sedimentation of solids and bacterial action.
- Wet ponds have positive impacts like providing a local wildlife habitat, improving on recreational and landscape amenities, which can translate into higher property values.

They are most effective in well-planned and organized residential communities with a reliable source of water.

**Constructed Wetlands**, these are shallow ponds and marshlands covered almost entirely in aquatic vegetation. They are established by transplanting live plants or dormant rhizomes from nursery stock or by seeding. They play an important role in pollutant removal and habitat improvement provided the design is such that surface area of the marshy part of the pond is maximized and a portion of inundated area is reserved for open water areas. The site for constructed wetlands must have sufficient base flow to maintain a relatively constant water level. (Schueler, 1987)

Storage can also take the form of **constructed underground reservoirs** into which water from roofs of households, or other building is directed. Open spaces within yards, driveways and footpaths can be used for this purpose. This stored water can be re-used in say toilet flushing, and/or aesthetic garden irrigation, when these reservoirs are filled with gravel they provide temporary storage prior to infiltration into the surrounding soil. (Pratt, 1999)

### 3.3.2 Wastewater drainage

Wastewater refers to dirt, soap, food, grease and bodily waste generated from households. It starts in kitchen sinks, toilets, showers/bathrooms, laundries, and often carries nutrients and pathogens.

#### A. Conventional options

Like in the case for storm drainage, wastewater has previously been conveyed through either separate or combined systems.

The **combined system** is as defined in section 3.3.1 above. The use of this system in tropical climates where intense storms are experienced is not desirable because then the excess surface and foul water flows would overflow into the receiving water bodies untreated having overwhelmed the treatment plant. In temperate climates where storm flows tend to be regular and therefore they can be accommodated with only slight increases in the sanitary sewers this kind of system is acceptable. (Stephenson, 2001)

The **separate system** consists of a set of pipes for wastewater flows from domestic and industrial sites known as waterborne sewerage (Phalafala, 2002). It is also termed deep sewerage and this stems from the fact that in actual practice the sewerage pipes are laid deep beneath the ground, which is in accordance with the conservative design assumptions on which govern the design procedure for this system. (Ho, 2000)

## B. Alternative options

The range of alternative technical options for wastewater drainage include; on-site management technologies and off-site conveyance systems.

### On-site wastewater management technology options

The ***Pour flush toilet (sewered or non-sewered)***, is a modification of the pit latrine and is widely used in India, South East Asia, and some parts of South America, where water is used for anal cleansing. The original design replaced the squatting plate of pit latrine with a water seal unit (pour flush bowl), the preferred design used in these countries, however, has a completely offset pit to which the pour flush bowl is connected by short length diameter pipe. The non-sewered type allows for the liquid portion to soak away into the surrounding soil while the solid portion undergoes anaerobic digestion (Ho, 2000). When adequate space is available, two pits should be built and used interchangeably, allowing for at least a one-year gap prior to emptying and reuse of pits.

The sewered type is an improvement to the aqua-privy, and comprises of the following elements; a short length of pipe which connects the pour flush bowl to a two-compartment watertight container, with the first compartment receiving excreta and flush water while the second receives only sullage. The first compartment settles out solids and discharges liquids into the second tank that is connected to a street sewer. (Kalbermatten, 1982)

The advantage of the pour flush toilet types include:

- They are free from odor, fly and mosquito nuisance and thus can be an in-house toilet, a social aspiration of many communities.
- These toilets are suitable even when water consumption is low and the sewered type applicable where sullage water disposal necessary.
- The pour flush toilet types can also be easily upgraded to a low cost sewerage system particularly when wastewater flow exceeds absorptive capacity of the soil.

***Septic tanks***, are rectangular chambers usually located just below the ground, which receive black water and grey water. In them solids and floating materials (e.g. oil and grease) are settled out and undergo anaerobic digestion producing sludge, while the liquid effluent is directed to a leach pit or trench similar to the pit of a pit latrine. The two-compartment tank is preferable to the single compartment one because concentration of suspended solids in its effluent is considerably lower. The pit must be sized to allow percolation of the volume of wastewater generated. This option best finds application in low-density upper class sub urban areas because of the high water service levels and space requirements. They can however, be

altered and made more suitable for medium density (i.e. 200-300 persons/ha) areas with design modifications and the use of water saving devices. When population density and water use increase beyond the absorptive capacity of the soils, septic tanks can also readily be upgraded by connection to small bore or conventional sewerage systems. (Kalbermatten et al, 1982)

With on-site wastewater drainage options the liquid portion of waste is infiltrated into the ground while the solid portion is stored in holding tanks and later removed as sludge, which is subjected to further treatment off-site before disposal on ground particularly as mulch in gardens. Other on-site technologies are VIP latrines, composting toilets and chemical toilets, however, they can only receive yellow and brown water (i.e. faeces and urine excluding flush water) therefore separate system would be necessary for grey water. They are widely used in rural areas of developing countries where water consumption levels are low and thus grey water is discharged into the yards by household dwellers. (Ho, 2000)

#### Alternative conveyance systems

The conventional sewerage systems described in *section 3.3.2, part A* are expensive because of the stipulations of minimum pipe and gradient requirements to minimize blockages, facilitate cleaning and ensure transportation of solids with flows in usual circumstances that result in bigger sized pipes (Stephenson, 2001). The following alternative conveyance systems have been developed following modifications to the design procedure for deep sewerage by adoption of design assumptions that are less conservative. This allows for shallower placement of pipes, and the use of smaller pipes when water use shown to be less. They are of two types:

***Small-bore sewerage (settled sewerage)***, this system was originally developed in the late 50's to improve the performance of aqua-privies in Northern Rhodesia (now Zambia). Other similar schemes have been implemented in Australia, Nigeria, South Africa and the US. The schemes in Africa are mainly upgrades to aqua privy or pour flush toilet systems, while those in Australia are upgrades to septic tank systems (Mara, 1996). This system is particularly suitable where on-site disposal has been practiced but cannot be continued without modification because: (Ho, 2000; Kalbermatten et al, 1982)

- Infiltration beds are no longer adequate,
- Clogged soak pits cannot be rehabilitated,

- The amount of sullage water has increased to the extent that onsite disposal no longer possible or
- The effluent pollutes the ground water

It is a low cost sanitation technology option with all the benefits of conventional sewerage with particular regard to capital investment requirements, public and environmental health protection. (Kalbermatten et al, 1982)

The settled sewerage system comprises a network of pipes designed to convey the liquid portion of sewage to a central treatment and/or disposal point while the solids in the sewage are settled out in an interceptor tank upstream of the pipe network, (Ho, 2000). It has therefore got both on-site and off-site elements, (Du Pisani, 1998). The conveyance of settled effluent only means that flatter slopes possible because scouring velocities to re-suspend settled solids are not necessary in a system that does not carry solids. Shallower pipe depths also possible because of flatter grades and the fact that effluent is discharged from settling tanks close to the ground surface (Kalbermatten et al, 1982)

Settled sewerage systems have existed in South Africa for over 10 years. There are reported to be at least 21 schemes serving 21 erven in high, middle and low income communities in the country. Capital cost savings of between 8.7 – 43% of the cost for installing conventional sewerage have been noted in the installation of these schemes, (Du Pisani, 1998).

***Simplified (shallow) sewerage***, this system was developed by South American Engineers in the early 80's in an attempt to provide an affordable sanitation alternative for dense urban settlements. It has been successfully implemented in Brazil, Greece, Australia, USA, Bolivia, India and Pakistan. It is also regarded as an extremely practical and low cost sewerage solution with the convenience and health benefits of conventional waterborne sanitation, which encourages social development within beneficiary communities (Eslick and Harrison, 2004). The environmental impact of shallow sewerage is also similar to conventional sewerage i.e. protecting water sources, people and the environment in general from human waste.

The shallow sewerage technology evolved following a relaxation of several design characteristics of conventional sewerage and is thus described as essentially conventional sewerage without any of its conservative design requirements i.e. it utilizes smaller diameter pipes, shallower depths and flatter gradients with fewer and simpler manholes (Mara, 1996; Eslick and Harrison, 2004).



The applicability of shallow sewer systems in South Africa has been investigated with the undertaking of a pilot study in the Briardale and Emmaus communities in the Ethekweni Municipality. The findings from this pilot study indicate that though shallow sewer systems provide an excellent sanitation solution in the “water and sanitation” package for South African communities, certain legal, technical and institutional issues need to be resolved, including:

- National building regulations particularly in regard to pipe diameters, sewer depths, access manhole requirements and construction methods are prohibitive.
- Conflict with land tenure principles in regard to the community ownership of the common sewer line, which is a social concept of the shallow sewer systems
- The need to structure service providers in a way that it can provide community-based services.

A few of the noted advantages of this technology following this study match the experience of other countries where this technology has been adopted (Mara, 1996; Eslick and Harrison, 2004). They are:

- 50% reduction in capital investment when compared to conventional sewerage, which also compares favorably to that of Ventilated Improved Pit latrines (VIPs), if costs are “ring-fenced” to the site of development (i.e. ignoring the capital costs of bulk reticulation and treatment works).
- Appropriate where water use is between 30 and 60 l/c/d which, is too high for VIPs and too low for conventional sewerage.
- Preferable to VIPs in denser formal and informal peri-urban settlements.

### **3.3.3 Grey water drainage**

Grey water is water from baths, showers, basins and other wash water. It is the largest volume of wastewater apart from toilet flush water generated by households.

#### **A. Conventional options**

Traditionally grey water has been directed into drains together with black water (i.e. toilet waste and flush water) and collectively referred to as sewage or household wastewater and conveyed through the separate or combined systems described in section 3.3.2 part A.

## B. Alternative options

The following alternative drainage options for the sullage waste stream are derived from the waste management hierarchy concept of separately collecting and managing waste streams so as to prevent irreversible contamination of wastes that can be used or recycled.

### Separate grey water drainage

In peri-urban areas where water consumption levels are higher than in rural areas and there is inadequate space for gardening, large grey water flows from households tipped into streets and alleys are posing a serious environmental and public health threat. The provision of grey water drains only for Kliptown in the greater Soweto area has had a marked improvement in the overall living conditions in the settlement, and resulted in a significant reduction in the load on the river (DWAF, 2001). This option is economically feasible because not only are the flows much lower because of the reduced peak load from toilet flushing, but also detention of peaks is very feasible, enabling discharge pipes to be relatively small bore. Gradients and depths are not as important as for conventional sewerage, and laying can be undertaken by relatively unskilled communities. Blockages are less likely, and screens and traps will remove any solids (Stephenson, 2001).

One major disadvantage with this conveyance system, however, is that it commits the community to non-water borne sewerage i.e. pit (in rural areas) or chemical toilets or vacuum tankers (in urban areas). Upgrading from small bore to large bore sewers would be difficult and expensive if water borne sewerage is desired, so careful thought and discussion is needed as to future plans. (Stephenson, 2001)

### Combined storm and grey water drainage

With this drainage system, grey water is directed into storm water drains. It is considered viable because separation at the downstream end and treatment of the concentrated stream is achievable requiring only simple treatment methods like skimming, settling and/or screening while separation would be by gravity. These two waste streams contain pollutants that can easily be removed using the methods described e.g. floating solids and oil, grit and larger solids like dead leaves etc. (Stephenson, 2001)

The main advantage of these alternative options is that they provide for economical off-site transportation system arrangements when quantities of flows from two of the major streams of concern exceed the natural capacity of in-situ soils.

The available range of conventional and alternative drainage technology options that can be considered in selection of appropriate drainage options for low cost townships are summarized in table 3.1 below. To aid comparison the advantages, disadvantages and requirements for successful application for each option are highlighted.

**Table 3.1: Available drainage options (Ho, 2000; Kalbermatten, 1982; Stephenson, 2001)**

<b>Drainage option category</b>	<b>Drainage option type</b>	<b>Advantages</b>	<b>Disadvantages</b>	<b>Requirements for successful application</b>
Conventional	Separate system	Convenient	Expensive to construct	Favorable site conditions i.e. neither too steep or flat topography
	Combined system	In temperate climates surface water can be accommodated with only slight increases in sanitary sewers	Susceptible to blockages where alternative cleaning methods used. Separation of sewage difficult when storm occurs Odorous in hot climates During dry periods separation and stranding of solids a problem.	Favorable site conditions i.e. neither too steep or flat topography
Alternative on-site disposal	VIP latrines	Easy to construct Cheap Hygienic	Doesn't allow for sullage disposal Expensive to empty, or require replacement	Not suitable if space is at a premium or not available. Not good if ground is rocky or ground water table is near surface

<b>Drainage option category</b>	<b>Drainage option type</b>	<b>Advantages</b>	<b>Disadvantages</b>	<b>Requirements for successful application</b>
	Composting toilet	Composts sewage solids for possible use as soil conditioner and fertilizer Odorless	Requires user attention Market for compost in urban areas may be scarce if not available on site of generation Doesn't provide for sullage disposal	Have large space requirements for each household is to accommodate alternate vaults.
	Pour flush (LOFLOS) toilets	Inexpensive Minimum water requirements, the use of sullage possible Odorless Can be upgraded to sewer	Possibility of ground water contamination. Water has to be carried to tank.	Similar to Pit latrines
	Septic tanks	Can handle both grey and black water Easy to upgrade to sewerage network if soil capacity to cope with effluent exceeded	Expensive to install O&M costs high Possibility of ground water contamination Not viable in dense urban settlements	Require substantial land area Not suitable in areas with shallow ground water table

<b>Drainage option category</b>	<b>Drainage option type</b>	<b>Advantages</b>	<b>Disadvantages</b>	<b>Requirements for successful application</b>
	Infiltration devices for storm water runoff reduction	Effectively remove particulate and soluble pollutants Help achieve ground water recharge, low flow augmentation and stream bank erosion control	Susceptible to clogging by sediments Not suitable for intensively developed sites	Require substantial land area Soil and geology must facilitate infiltration
Alternative conveyance systems	Separate grey drainage	Inexpensive Easy to install	Restricts beneficiary community to non-water borne sewerage	
	Combined storm and grey water systems	Simplify the treatment and separation of concentrated stream	Significantly more expensive than the separate grey water drainage because of high storm flow rates	
	Simplified sewerage	High density development not a limitation Cheaper option than the separate sewerage option under conventional approaches. Can be used in areas with		Economies of scale make it more economical in dense urban areas

<b>Drainage option category</b>	<b>Drainage option type</b>	<b>Advantages</b>	<b>Disadvantages</b>	<b>Requirements for successful application</b>
	Settled sewerage	<p>medium water consumption</p> <p>Useful in areas where soil can no longer cope with effluent from septic tanks.</p> <p>In expensive when compared to conventional separate foul sewerage</p>	<p>O&amp;M requirements high</p> <p>Not suitable for high density sites</p>	Requires substantial land area

### **3.4 DRAINAGE OPTION SELECTION**

The choice of appropriate drainage option from the range of options described in section 3.3 above, for application within a given context, requires an assessment of the following factors: (Wright, 1986; Schueler, 1987; Pegram and Palmer, 1999)

- **Local site conditions e.g.**
  - Soil and geology conditions
  - Climate
  - Topography
  - Housing and population densities i.e. space availability
  - Water supply service levels
  - Property values
- **Socio-economic factors e.g.**
  - Affordability
  - Willingness to pay for services
  - Social acceptance
  - Income levels
- **Institutional factors**
  - Level of technical skill within beneficiary community
  - Stability and organization of the community
  - Human resource capacity of relevant municipal service provider
- Financial resources to facilitate capital investment of infrastructure and funds for routine and non-routine operation and maintenance procedures of infrastructure installed.





































A preliminary step in selection of the appropriate technology option is to assess the constraints that the local site conditions place on the range of options. This way the list of applicable options is shortened by elimination of those options that are physically not feasible for the site.




### **3.5 PRELIMINARY SCREENING OF OPTIONS**

The drainage technology options described are screened against the specific site conditions in Alexandra west bank, the case study area, in figures 3.1 and 3.2 below. Figure 3.1 is specific to the wastewater drainage options while figure 3.2 deals with the storm drainage options.



Figure 3.1 Feasibility of the available wastewater drainage technology options given the Alexandra physical site condition situation. (Schueler, 1987)


























Technology option	high densities i.e. 227.18hh/ha	yard tap minimum level of service, 3 taps per stand	medium slopes i.e. 5% - 12.5%	perched water table in some locations within < 1 m of GL, otherwise ranges 1.3-7.8m depth	sandy clay or clayey sand soil types overlain by hilly and gully wash; i ranges from 1-4mm/hr	depth to bedrock up to 9m
ON-SITE MANAGEMENT						
VIP Latrines						
Pour flush (LOFLOS) toilets						
Septic tanks						
OFF-SITE MANAGEMENT						
Settled sewerage						
Simplified sewerage						
Conventional sewerage						

 **PHYSICAL CONDITION A LIMITATION TO THE USE OF TECHNOLOGY OPTION**  
 **SITE CONDITION NOT A LIMITATION TO THE USE OF TECHNOLOGY OPTION**  
 **PHYSICAL CONDITION A LIMITATION TO THE USE OF TECHNOLOGY IN SOME PARTS OF SITE**

From figure 3.1 above, it is seen that the simplified sewerage option best fits the physical site conditions in the case study area, however, settled sewerage and conventional sewerage could be appropriate if the necessary adjustments are made to the housing density and water supply service levels. In the future development plans outlined for Alexandra, reduction in population densities is not expected to exceed 10% of current population, which rules out the application of settled sewerage. In the case of water supply service levels, the existing communal facility arrangement is to be retained, with a 1:6 residents per facility, which would likely increase water supply consumption, as to whether this would equal or exceed the 75l/c/d requirement for installation of a conventional system would need to be examined through field surveys. The conventional sewerage option has the disadvantage of being

prohibitively expensive, and thus more a last resort when dealing with low-income settlements.

Figure 3.2: Feasibility of alternative storm drainage options given the physical site conditions in Alexandria

technology option	high densities at 227.18hh/ha i.e. limited space	medium slopes i.e. 5% - 12.5%	perched water table in some locations within < 1 m of GL, otherwise ranges 1.3-7.8m depth	sandy clay or clayey sand soil types overlain by hilly and gully wash; i ranges from 1-4mm/hr	depth to bedrock up to 9m
<b>Retention/Detention facilities</b>					
Dry ponds					
Wet ponds					
<b>Infiltration facilities</b>					
Infiltration trench					
Infiltration basin					
<b>Vegetative BMPs</b>					
Constructed wetlands					




 **PHYSICAL SITE CONDITIONS A LIMITATION TO THE USE OF TECHNOLOGY OPTION**  
 **PHYSICAL SITE CONDITIONS NOT A LIMITATION TO THE USE OF TECHNOLOGY OPTION**  
 **PHYSICAL SITE CONDITIONS A LIMITATION TO THE USE OF TECHNOLOGY OPTION IN CERTAIN SECTIONS OF THE STUDY AREA**

Figure 3.2 above shows that in regard to the specified site conditions in Alexandria all the alternative storm drainage options are limited by the high densities. The dry pond and infiltration BMPs are especially constrained because of the poor draining properties of the in-situ soil conditions and the perched water table in certain locations of the site.

### 3.6 SUMMARY OF LITERATURE REVIEW FINDINGS

The findings on available technologies for wastewater and storm water runoff control indicate that apart from the conventional methods that are financially prohibitive and also don't take

into consideration the resource value of the drainage flows from low income peri urban settlements, the alternative methods are mainly storage (sometimes for reuse) and infiltration (for ground water recharge) techniques that involve managing the drainage flows at the point of generation.

These techniques, however, have large space requirements, and/or are subject to the infiltration rate afforded by the soil types at the site of interest. Alexandra west bank, case study area for this research study, is unfortunately too densely developed and even with the 10% reduction in population densities planned, this wouldn't result in significant space availability. In the case of infiltration, the soil type infiltration rate ranges from 1- 4mm/hr, which is outside the range recommended for infiltration type practices.

The conclusions from the preliminary screening are therefore narrowing down to the traditional approach of off-site drainage practices. In regards to the wastewater component of the drainage flows, the simplified sewerage option, one of the alternative off-site transportation systems to the conventional/deep sewerage option feasible. In regards to managing storm water runoff, social and institutional measures like public education, street cleaning, storm drain flushing would result in more effective performance of existing storm drains, as they are all designed to handle flows of up to the 10-yr flood level, the limit normally designed for in predominantly residential areas. The major floods can be handled by way of utilizing the roadways and the natural drainage pathways to be opened up following de-densification that will be targeting residents currently occupying the tributaries of Jukskei.

## **CHAPTER FOUR: DRAINAGE FLOW ESTIMATES**

### **4.1 INTRODUCTION**

In this chapter, the focus is on estimating the quantities of drainage flows (i.e. grey water, black water and storm water) emanating from Alexandra proper settlement. The results of the flow estimation are used in determining the capacity of outfall pipes for proposed arrangements of drainage systems. The procedures for determining quantities of the component waste streams comprising drainage flows from the study area are discussed separately.

### **PART A: SEWAGE FLOW ESTIMATES**

#### **4.2 BRIEF BACKGROUND**

The discussion below is a report on the procedures followed in determining the proportion of wastewater flows emanating from Alexandra proper drainage area that constitute sewage i.e. “grey”, and black water

#### **4.3 SEWAGE FLOW ESTIMATION METHODS**

The proportion of water produced that reaches the sanitary sewer system from a given drainage area can be determined by analyzing data from any of these sources: (Metcalf & Eddy, 1991)

METHOD A. Water supply and consumption records for the service area and their relation to wastewater flow rates.

METHOD B. Direct field measurements of actual sewage flow rates from the area taken at strategic locations within the sewerage reticulation network.

METHOD C. Sewage flow estimations using typical wastewater flow rate data from different establishments under the different categories of land use types.

##### **4.3.1 Estimating flows from water supply and water use data (Method A)**

This method involves the application of appropriate percentages to records from metered water use, to obtain a reasonable estimate of wastewater flow rates. It is employed when field measurements of wastewater flow rates are not possible and actual wastewater flow rate data are not available. The estimate will exclude the amount contributed by extraneous flows.

#### **4.3.1.1 Data requirements**

1.The use of this method requires information on the water use data of the different consumer categories within the municipal urban area, and they normally include: (Metcalf & Eddy, 1991)

- Domestic water which is water supplied to:
  - Residential areas,
  - Commercial districts,
  - Institutional facilities and
  - Recreational facilities
- Industrial i.e. non domestic purposes
- Public services e.g. fire fighting, system maintenance and municipal landscape maintenance.
- Unaccounted for system losses and leakages

The water use data is normally obtained from records of bulk and/or individual water meter measurements installed for each consumer category over a period of time that show seasonal and other variations. In the absence of actual water meter measurements, the water use can be estimated from typical per capita values developed for the different establishments falling under the above water use categories. The reliability of the metered records is dependent on the accuracy of meters installed to record flows and this must be determined.

The use of typical per capita values should be based on the similarity between the conditions under which they were developed and those prevailing in the service area for which wastewater flow rates are being estimated. This is because variation in water use has been observed in different communities depending on the following factors:

- Climate e.g. water use found to be at its peak when it is hot and dry.
- Proximity to the water supply service determined by service level under which consumers in the service area are served. It has been found that those with in-house water supply connections use more than those who have to walk some distance to a water source.
- Dependability and quality of supply will encourage use by consumers while supplies that are not dependable in terms of poor pressure and limited quantities have lower water use.
- The lack of or practice of encouraging water conservation and installation of water saving devices.

- Metered and billed services found to prevent waste of water by users reducing actual water use while metered or un-metered but not billed service encourages waste.
- Demographics of the study area especially land use types (particularly the predominant type), population & housing densities.
- The income bracket of the water supply consumers in the service area under study impact on the water supply service levels affordable to them and thus their ultimate consumption levels.

The water consumption for the service area under study as obtained from either water use data kept by the relevant agency or estimated using typical per capita values is compared with records on water produced or withdrawn and discharged into the water supply system as taken from bulk meters installed at the point of withdrawal for supply. The purpose for this is to determine what constitutes the water lost or unaccounted for in the distribution system as this component does not reach the wastewater system and must be excluded from sewage flow estimates. The other portions of water used that do not reach the sanitary sewers that must also be excluded from estimations include:

- Product water used by manufacturing establishments
- Water used for landscape irrigation, system maintenance and extinguishing fires
- Water used by consumers whose facilities not connected to sewers

2. Appropriate peak factors for application to typical average values of the water used.

#### **4.3.1.2 Data availability**

In the case of Alexandra, the study area for this research, the political, security, socio-economic and other factors previously and currently prevailing in the area has limited the ability of relevant institutions to monitor and collect the relevant information. The only information that could readily be obtained was bulk water meter flow measurements recorded during the period July 2002-October 2003. The accuracy of these records is suspect as the meters were found to be malfunctioning during a flow monitoring study on the reticulation system for the area undertaken by Goba Moahloli Kevwe Steyn (GMKS) from 11/09/02 to 18/09/02. In addition, many of the existing individual consumer categories are not metered, while for those that are metered, meter readings are not taken as community representatives still in the process of introducing the culture of payment for services to the consumers.

The report on *GUIDELINES FOR HUMAN SETTLEMENT PLANNING AND DESIGN (Red book)*, (CSIR, 2000) outlines typical per capita water use values for different establishments, however they are generalized and not representative of the prevailing situation in low cost townships like Alexandra, which has varying levels of service due to its mixed formal and informal nature.

#### **4.3.2 Direct flow measurements taken at strategic locations within the sewer reticulation network, (Method B)**

This method involves identification of principal sources of wastewater within the area under study and the undertaking of direct flow measurements at strategic manhole locations close to them along the sewer reticulation network serving them, (Metcalf & Eddy, 1991). In addition, flow measurements are also undertaken at strategic locations along major interceptor and collector mains serving sub catchments within the study area sub divided according to land use types, water supply service levels and population densities.

The common principal sources within an area are residential areas, commercial districts, institutional, and recreational facilities. The measurements are taken over or during periods of time when fluctuations due to seasonal and other variations are also monitored.

The flow measurements obtained are inclusive of extraneous contributions from other sources. The extraneous contributions must be established and excluded before the specific flow rates for each source can be determined. The following data requirements are necessary to eliminate contributions from extraneous sources.

##### **4.3.2.1 Data requirements**

The method requires information on:

1. Layout of existing sewerage reticulation networks serving the area, from which strategically located manhole positions can be selected for direct measurement of flows.
2. The type of sewer system used for the removal of storm water and wastewater. These are the common types: (Metcalf & Eddy, 1991)
  - Separate sewers i.e. sanitary sewer systems and storm water sewers
  - Combined sewer systems

In the case of Alexandra west bank, as in the rest of South Africa, separate systems are used in the conveyance of storm and sanitary wastewater.

Information on the different components that make up the sewer inflow measured from the manholes and their proportionate amounts. The different components include: (Stephenson, 1988; Metcalf & Eddy, 1991)

- Sewage inflow, which comprises wastewater flow from the manholes in close proximity to the principal, sources being monitored and/or the total wastewater flows that reach the outfall sanitary sewer from the entire study area.
  - Storm water ingress that enters the sewer system through gullies, and manhole covers.
  - Steady ground water infiltration and leakages that enter the system through sewer service connections in the ground through such means as defective pipes, pipe joints, connections or manhole joints.
3. The estimates of the percentage contribution to sewage inflow by ground water infiltration (GWI) and leakages, as well as storm water ingress are described as follows:
- GWI is found to range between 0.0094 – 0.94 m<sup>3</sup>/d.mm-km or more depending on the age of sewers and ground water table levels. (Metcalf & Eddy, 1991)
  - Storm water ingress is assumed to be about 1% of precipitation over the catchment, however actual values depend on the methods of controlling storm water inflow into gullies. A percentage increase ranging between 50 – 69% was observed in sewer flows after a storm. (Stephenson, 1988)

#### **4.3.3 Sewage flow estimations using typical wastewater flow rate data from different establishments under the different categories of land use types, (Method C)**

This method involves estimating sewage flow rates for the case study area using typical wastewater flow rate data proposed for each of the principal sources of wastewater defined in method B above.

##### **4.3.3.1 Data requirements**

This method requires that typical unit wastewater flow rate data from each of the principal wastewater sources identified in the area is known. The peak flow rates are normally expressed in liter per min for the establishments falling under the different categories of land use types.



#### **4.3.3.2 Data availability**

The information that could be readily obtained from existing records for application of this method of sewage flow estimations is outlined below: (Stephenson, 1988)

Unit peak wastewater flow rates for the major principal sources in the Johannesburg area including:

- Low and high-income residential areas
- Commercial areas
- Apartment (flats) areas and
- Industrial areas

#### **4.4 ADOPTED METHODS AND JUSTIFICATIONS**

For the purpose of this research study method B is adopted for estimation of sewage flows from Alexandra West bank catchment. Method A could not be used because water supply and consumption data sets necessary for application of this method are either non-existent and/or doubt has been cast on the accuracy of the limited data that can be accessed. In regards to method C, there is not enough detailed data on demographic, land use, different development type structures for the various land use categories and other cadastral data to apply this method.

#### **4.5 FIELD MEASUREMENTS**

With the adoption of field measurements as the most feasible method for determining domestic wastewater flows generated in Alexandra proper, the means of disposing wastewater generated in Alexandra were identified to assist in selecting appropriate measurement locations. They include:

- Water borne sewerage system accessible to formal residential development, (i.e. flats, apartments, hostels, original formal houses) institutions (i.e. schools, churches and clinics) and community recreational facilities (i.e. stadium, community center)
- Communal ablution blocks serving backyard shack developments on the original formal house stands. These are also connected to the water borne sewer infrastructure.
- Chemical toilets, approximately 720 of them, which are a temporary sanitation intervention for shack dwellers without access to ablution blocks. They designed to handle only the faeces and yellow water component of domestic wastewater.

- Storm water culvert drains are also informally being used to dispose of sewer effluent particularly the sillage generated by shack dwellers using chemical toilets.

The field measurements undertaken were therefore at convenient locations on the major sewer collector mains serving those connected to the water borne sewerage system, and the storm water culverts. In addition, the water use and wastewater characteristics of occupants of three backyard shack dwellings served by communal ablution facilities were observed.

#### **4.5.1 Measurements at manhole stations along the major sewer collector mains**

##### **4.5.1.1 Positions of flow measurements**

During the month of November 2003, Potgieter Hattingh and Raspi (PH&R), the consulting engineers responsible for determining the extent of upgrading and rehabilitation of sewer infrastructure works required in Alexandra proper undertook flow measurements at five manhole stations using the Flocap Model 2 ultrasonic level measurement meters. (PH&R, 2003) These manhole stations were along the major collector and outfall sewer mains serving five of the seven major sewer sub-catchments, see figure 4.1 below. The sub-division of the Alexandra drainage area made use of the natural catchment topography and existing major roads as boundaries, as seen in figure 4.2.

The same manhole locations were selected for manual measurements taken during the winter month of June. *PART F512.14* of the *SEWER DESIGN MANUAL FOR THE CITY OF LOS ANGELES* recommend that for existing sewers, the procedures for measurement of flow should entail depth of flow readings at ½ or 1 hour intervals for not less than 24 hrs. The manual measurements taken during the month of June 2004 were done for 24 hours over a three-day period, (Los Angeles City, 2003). Budgetary constraints and site limitations contributed to the decision to take manual and not metered (i.e. using manufactured sewer flow meters) measurements.

##### **4.5.1.2 Procedures followed in carrying out field measurements**

The depth of flow at each of the manhole stations was measured manually at 1-hr intervals for 24 hours over a period of 3 days. The 24-hr measurement was selected to enable accurate estimates of average flow rates and peak factor values from the observed maximum average flows.

##### **4.5.1.3 Equipment**

- Small diameter rod of 4m-length dipped in flowing sewage, from which depth of flow could be taken off.
- 5m and 30m tape used in taking readings off wetted portion of dipped rod.



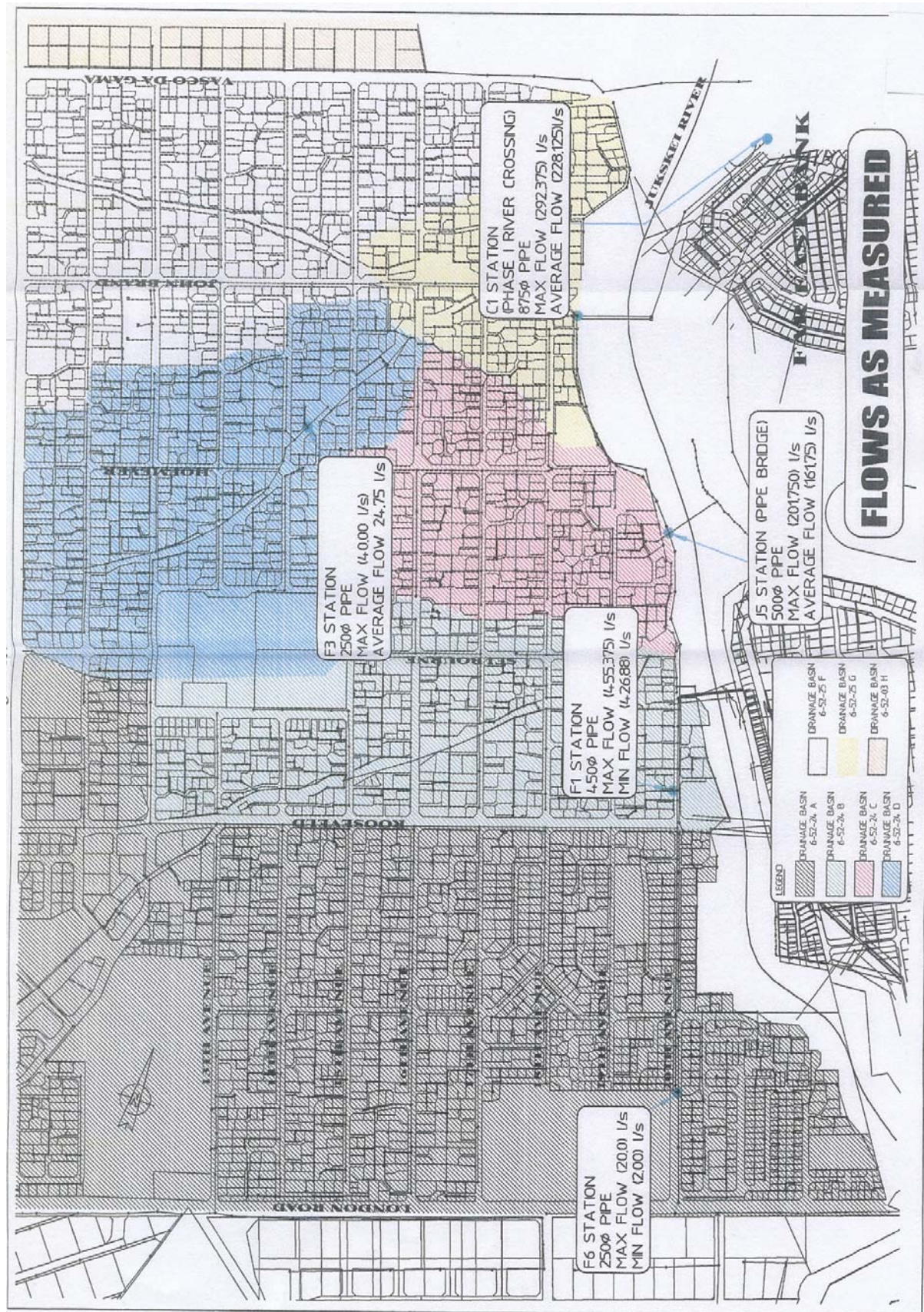


Figure 4.1: Location of flow monitoring manhole stations along sewer collector mains serving the sewer sub-catchments



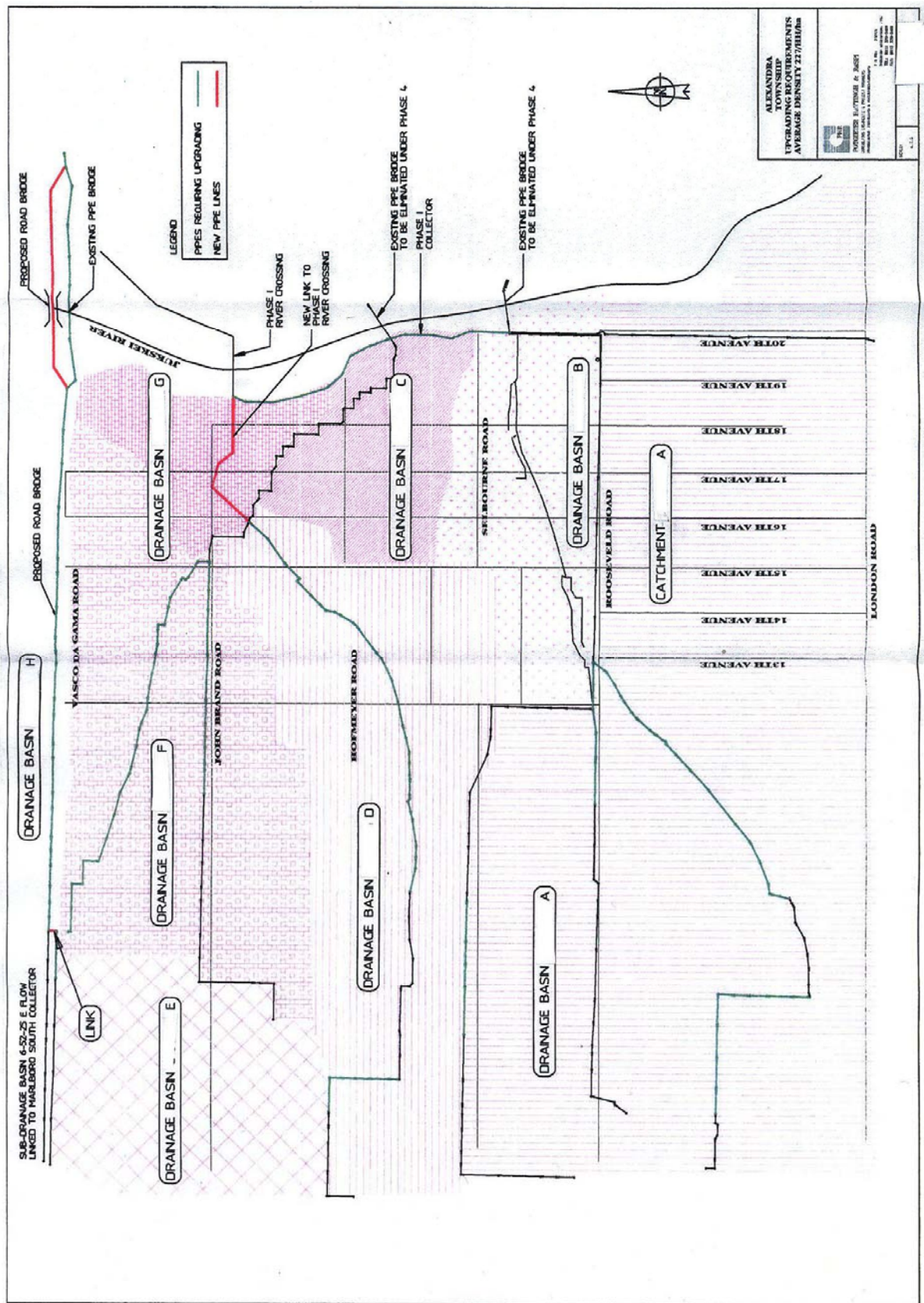


Figure 4.2: Sewer sub-catchments of Alexandra west bank

## **4.5.2 Measurements of flows from storm water drainage system**

### **4.5.2.1 Positions for flow measurement**

The storm water culvert drainage system for Alexandra proper is being informally used (directly and indirectly) for domestic wastewater disposal by shack dwellers without adequate disposal facilities. At the outlets of all the three storm water culverts, temporary diversion structures and pipelines have therefore been erected to divert the flow into the sewer collector and outfall mains for the area until the sanitation situation is improved.

Two manhole locations located along diversion pipelines from the outlets of the culverts were identified for depth of flow measurements. These locations are not detailed on layout and profile drawings of the sewer infrastructure because they are temporary, and so the pipe dimensions and slopes used in computation of flow from depth measurements are fairly accurate estimates determined using a survey staff. The general location is marked on the figure 4.3 below.

### **4.5.2.2 Procedure**

The procedure followed in carrying out the depth of flow measurements for the collector main serving specific sewer sub-catchments was used in this case as well.

### **4.5.2.3 Equipment**

- Survey staff –used in estimating pipe diameters
- Small diameter rod of 4m-length dipped in flowing sewage, from which depth of flow could be taken off.
- 5m and 30m tape used in taking readings off wetted portion of dipped rod



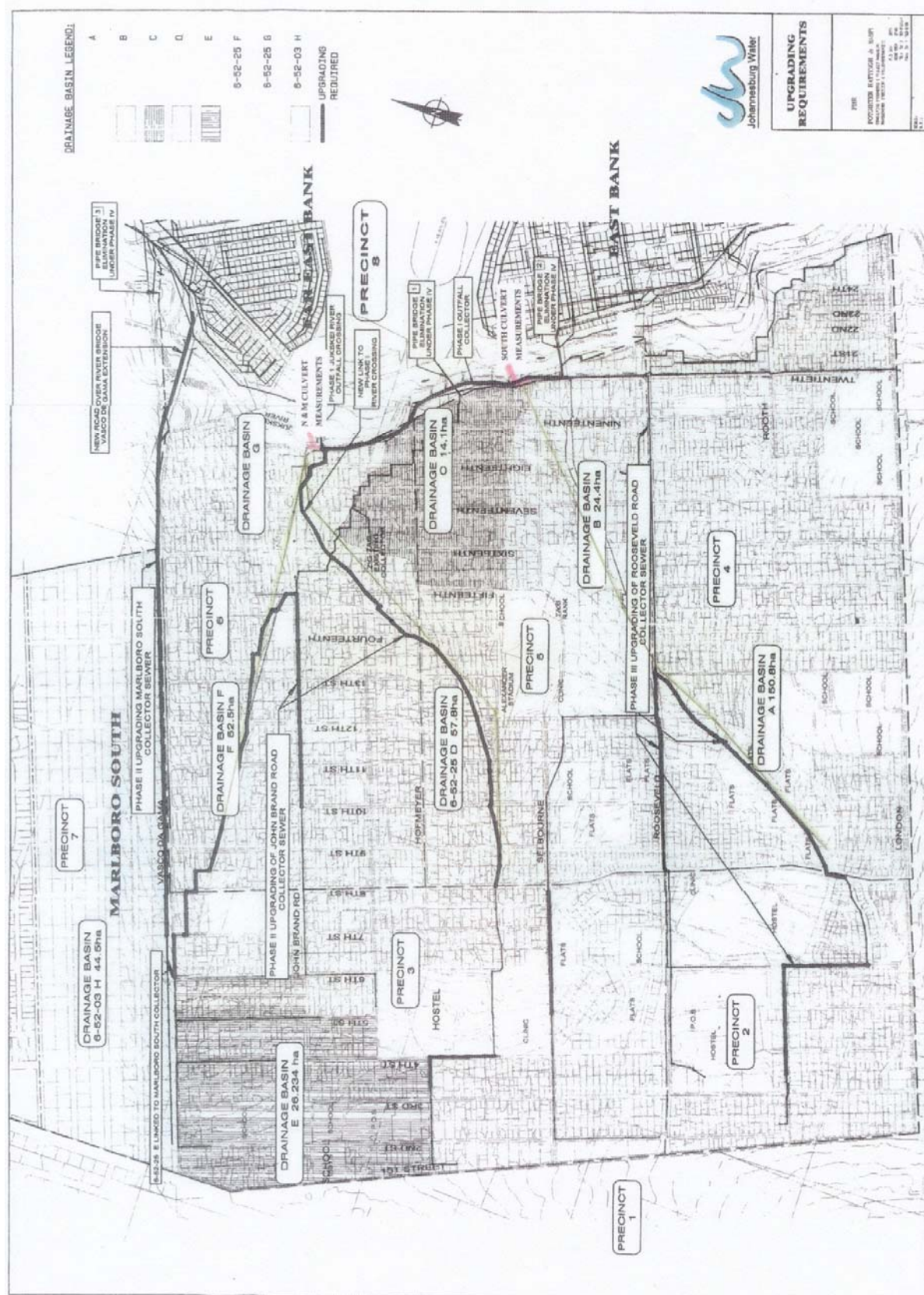


Figure 4.3: Location of flow measurements stations along the storm culvert drains

### **4.5.3 Field observations on the water usage and wastewater disposal characteristics of backyard shack dwellers**

The water use and wastewater disposal characteristics of occupants of three shack dwellings in the backyard of an original formal house stand along the eighth avenue between Vasco da Gama and John Brand streets sharing communal ablution facilities were observed over a period of one week. This became necessary because the findings on particularly consumption levels of residents as derived from the responses to a questionnaire survey administered in August 2003 were not convincing. (See appendix B)

These close up observations also provided opportunity to establish the purposes for which water is used in the peri-urban setting making it possible to approximate the percentage value of the grey and black water components of the domestic wastewater. The fraction of water consumed by residents that is returned as wastewater, (known as the return factor) could thus be reasonably determined, considering that water supply and use data is not readily available. The figures quoted in the red book guidelines for the provision of Engineering services and amenities in residential township developments have been determined using water supply and consumption data from areas that have characteristics that differ from those in peri-urban settlements.

#### **4.5.3.1 Procedure**

The water usage and wastewater disposal practices of the residents of these three backyard shack household units were observed throughout the day up to the time they retired for the day. Note was taken of the variation in water consumption at specific times of the day, the purposes for which water was used and the number of people present in each household at the specific times.

#### **4.5.3.2 Equipment**

To ensure that observed estimates were accurate, the residents of the three households were provided with 300ml cups/glasses to utilize when drawing small quantities of water e.g. drinking. The container used by the residents to fetch water from faucets was a 25-liter bucket.

## **4.6 FIELD MEASUREMENT FINDINGS AND THE INTERPRETATION**

### **4.6.1 Sewer collector and outfall main flow measurements**

The findings of sewer flow measurements undertaken at two manhole stations on the collector mains serving Alexandra proper as recorded by the PH&R data-logging instrument in November 2003, as well as manual measurements in June 2004 at the same stations are summarized in the table below.

Table 4.1: Summary of flow measurements from major sewer sub-catchments in Alexandria proper

Manhole station	Sewer sub-catchment monitored	Flow rates for Nov. 2003 in l/s		Flow rates for June 2004, l/s	
		ADF	Max. ADF	ADF	Max. ADF
F1	A	298.8	318.8	261.7	302.5
J5	C, D, E & F	113.2	141.2	91.1	123.6

These two stations are along collector mains serving sewer sub-catchments A and C, D, E & F (see figures 4.1 & 4.2 prepared by PH&R that shows the sewer sub-catchments of the study area and the location of these manholes). These two stations are selected because they serve the sub-catchments that cover a great part of Alexandria, and also represent the mixed formal/informal nature of its development.

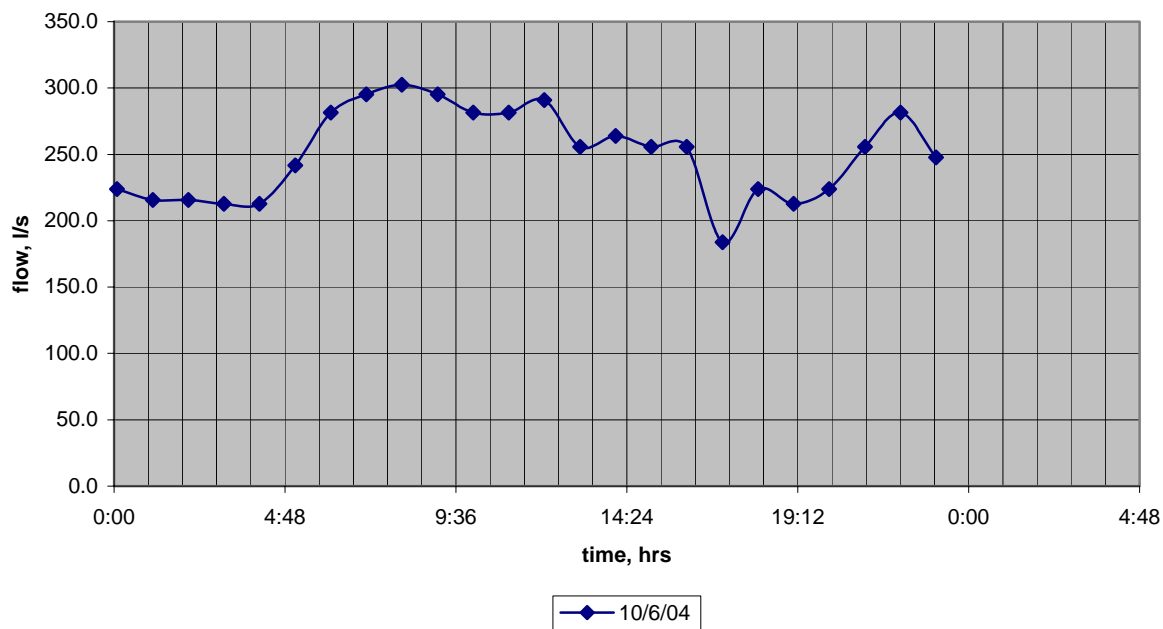
The flow measurements undertaken in November 2003 by PH&R were derived from measurements logged using a Flocap Model 2 Ultrasonic level measurement meter by taking into account the upstream inlet pipe slope, a roughness coefficient of 0.013 and the depth of sand/silt present, and are shown in table 4.1 for the manhole stations specified.

The theory of pipes flowing partially full was used in computing from the flow depths manually measured in June 2004 the flows through the sewers. The chart relating the ratio of the measured flow depth and sewer diameter on the vertical axis to the ratio of partial discharge and full pipe capacity on the horizontal axis was used for this purpose, this chart is reproduced in appendix C. These flows are considered to exclude the storm water ingress component of sewer inflow, since rains not recorded on general Johannesburg area over the last two months.

The graphical plots of the flow rates as logged by PH&R instrumentation during the November 2003 measurements, and those derived from depth of flow measurements for June 2004, calculated using the excel software program for similar days are presented in figures 4.4 (a)-(d) below for comparison. The spreadsheets showing calculations of these flows are attached in appendix C.



**Figure 4.4(a): Flow measurements for stn F1 (undertaken in June 2004, as part of data collection for research study)**



**Figure 4.4 (b): Flow measurements for stn J5 (undertaken in June 2004, as part of data collection for research study)**

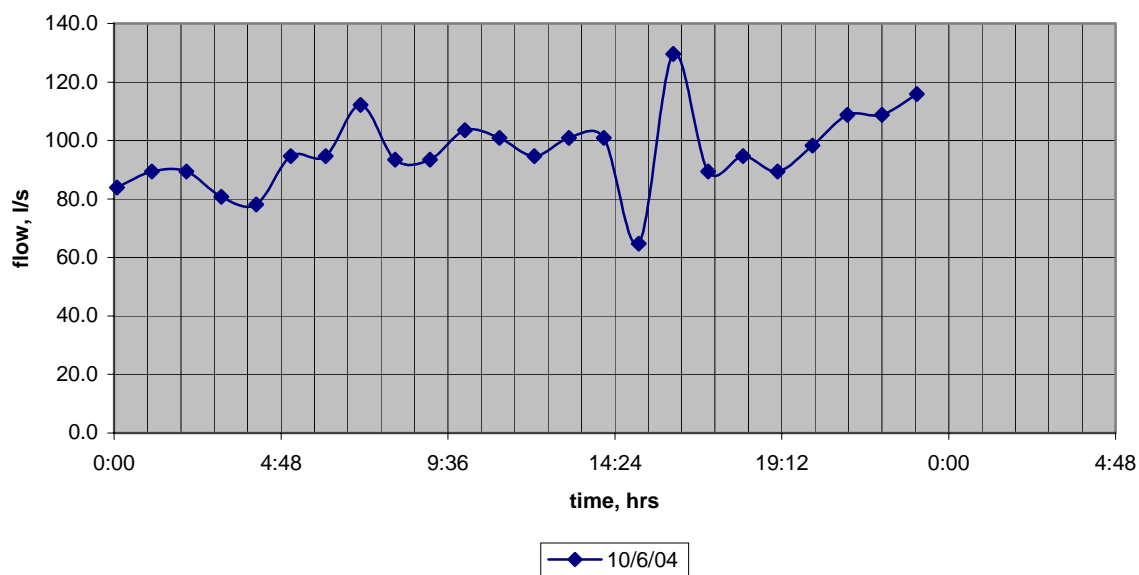
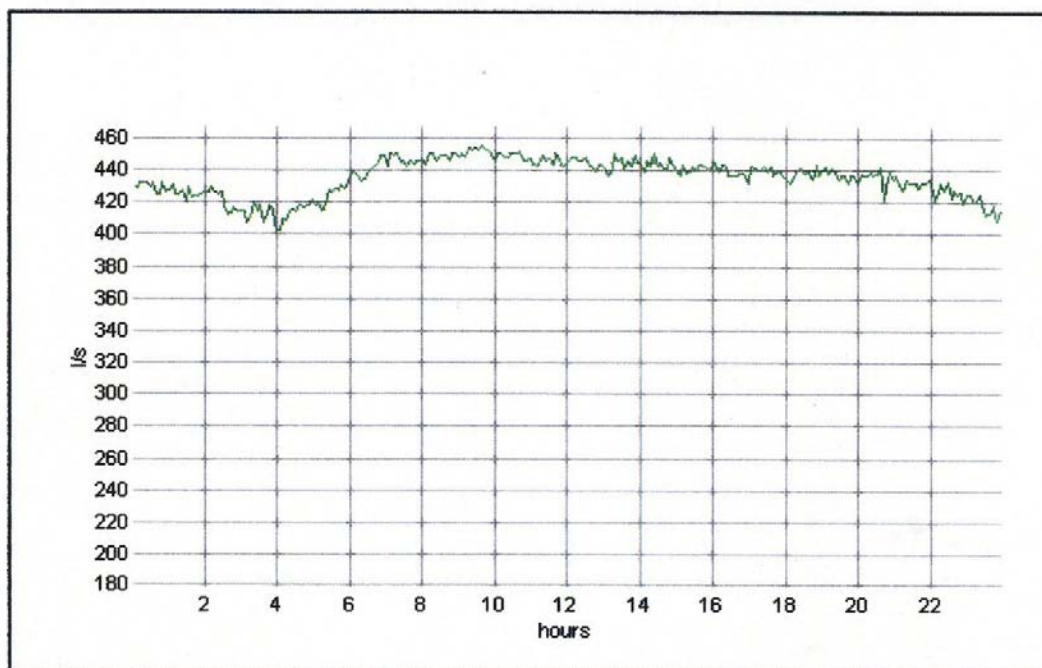


Figure 4.4 (c): Flow measurements for station F1, as recorded by the meter installed by PH&R in November, 2003.

Station:	:	F1	Max Flow	:	455 l/s
Period	:	2003-11-16 00:01:54.2	Min Flow	:	0 l/s
Total Flow	:	37556297 litres	Avg Flow	:	435 l/s



Station:	:	J5	Max Flow	:	215 l/s
Period	:	2003-11-15 00:01:58.8	Min Flow	:	0 l/s
Total Flow	:	14645421 litres	Avg Flow	:	170 l/s

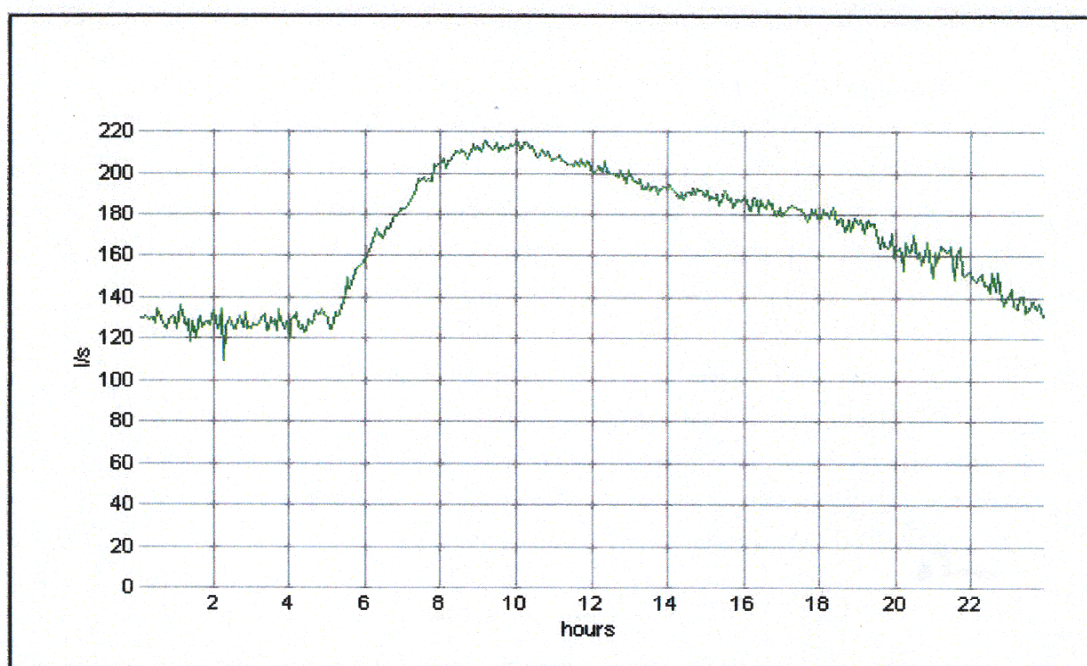


Figure 4.4 (d): Flow measurements for station J5, as recorded by the meter installed by PH&R in November, 2003.

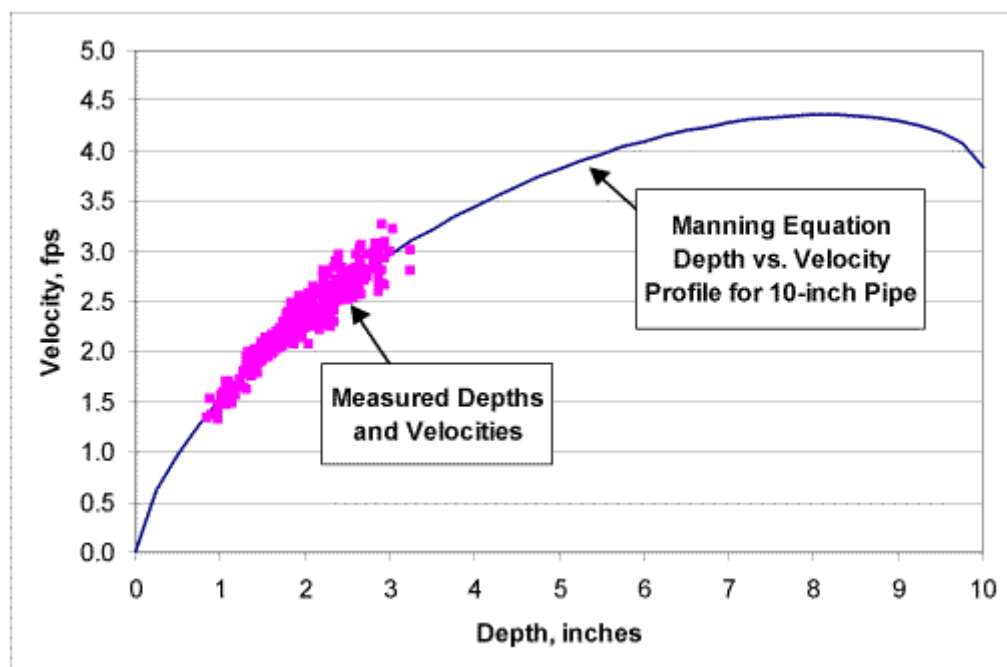
The plots of sewer flow shown above indicate a high night flow component, which can be attributed to leakages from faulty taps and plumbing fixtures installed in communal ablution facilities and individual households directly connected to the waterborne sewerage system. Appendix A4 shows photographs of taps for laundry troughs of the communal ablution facility left running by residents, and thus wastage could also be another reason for the high night flows.

#### **4.6.1.1 Accuracy of flow data taken along sewer collector mains**

The accuracy of sewer flow data can be ensured through various methods, including:

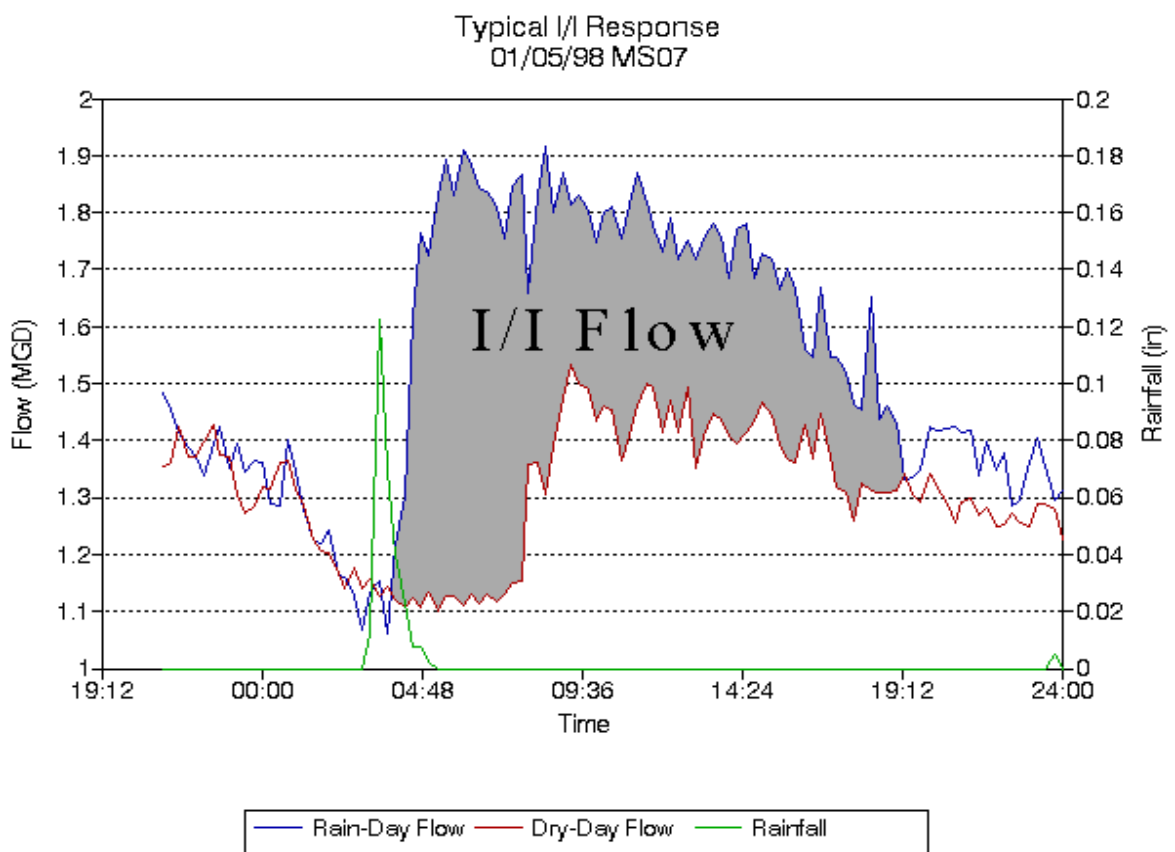
- 1) Calibration of flow metering equipment using the dye dilution, salt dilution, or manual measurements using ruler and portable velocity meters. This is preliminary field-testing to establish the percentage error of flow readings taken using instrumentation. (McBirney, 2005)
- 2) Velocity and depth relationships for a multiple range of readings taken. This relationship for free flowing gravity sewers takes the form of a concave downward pipe curve of similar shape to the theoretical Manning curve. In figure 4.5 shown below, is a velocity-depth plot for a 250mm diameter pipe derived using the Manning equation. (Stevens, 1998)

Figure 4.5: Characteristic velocity-depth plot for free flowing sewer



- 3) Depth-discharge scatter graphs, which display the calculated flow rate at each depth reading. The pattern of this curve for free flowing gravity sewers is similar to that of the velocity-depth relationship, but it is concave upwards. (Stevens, 1998)
- 4) The hydrographs generated from wastewater flow data follow predictable patterns or trends, and thus the hydrograph generated from measured sewer flow data can be compared with the classic diurnal hydrograph to establish the reliability of data collected. The classic diurnal pattern for wastewater flow is shown in figure 4.6 below. This method is not applicable where upstream conditions may result in irregular flow patterns e.g. if measurements recorded downstream of industrial operations.

Figure 4.6: Typical diurnal pattern for wastewater flow, depicting response during dry and wet weather periods (Wade, 2002)



For the sewer flow data obtained from measurements carried out in June 2004 at the specified locations on the Alexandra sewerage reticulation, only methods 3 and 4 were applicable for the following reasons:

#### Site restrictions

The haphazard development in Alexandra has resulted in informal shack dwellings being erected atop storm water drainage and sewerage infrastructure, particularly over grid inlets and inspection manholes. This made it difficult to find accessible adjoining manholes for the section of sewer reticulation where measurements were taken. Manual velocity of flow measurements would require at least two adjoining manholes. The absence of velocity of flow data makes the plot of velocity and depth relationship impossible.

#### Time and budgetary constraints

Insufficient funds and inadequate time could not allow for purchase of instrumentation for use in taking sewer flow measurement, and thus the need to adopt manually taking depth of flow readings at the manhole locations specified. This negated the need for calibration of instruments. The site restrictions discussed above also would have resulted in difficulty in using the dye dilution and salt dilution methods, which also require at least two adjoining manholes to be carried out.

In the figures 4.7(a) and (b) below, the depth-discharge relationships for the sewage flow data taken in June 2004 at stations F1 and J5 are shown.

**Figure 4.7(a): Depth-discharge plot, F1 - 10/06/04**

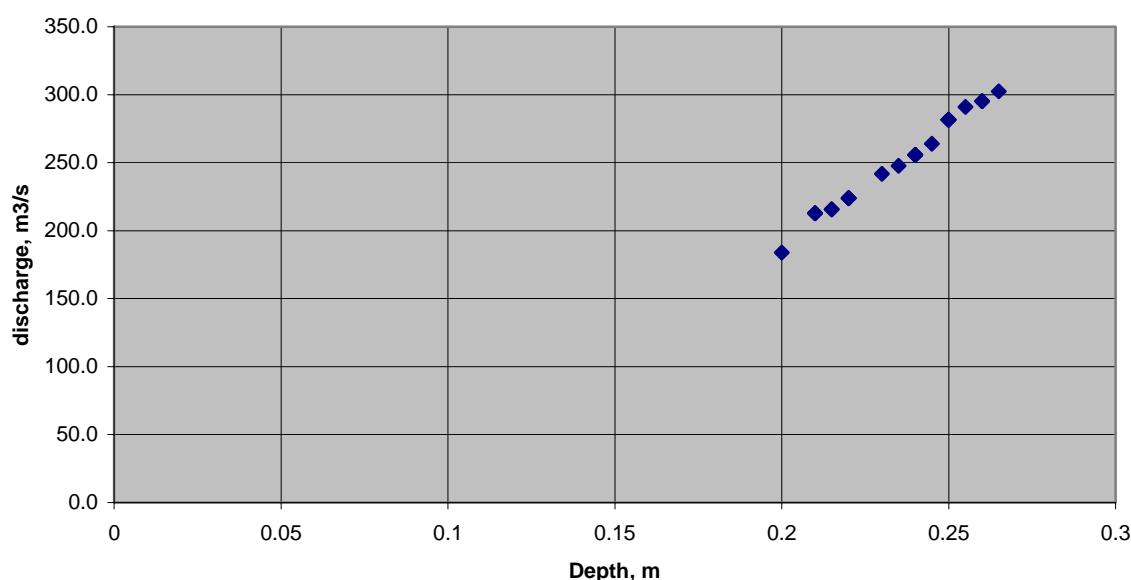
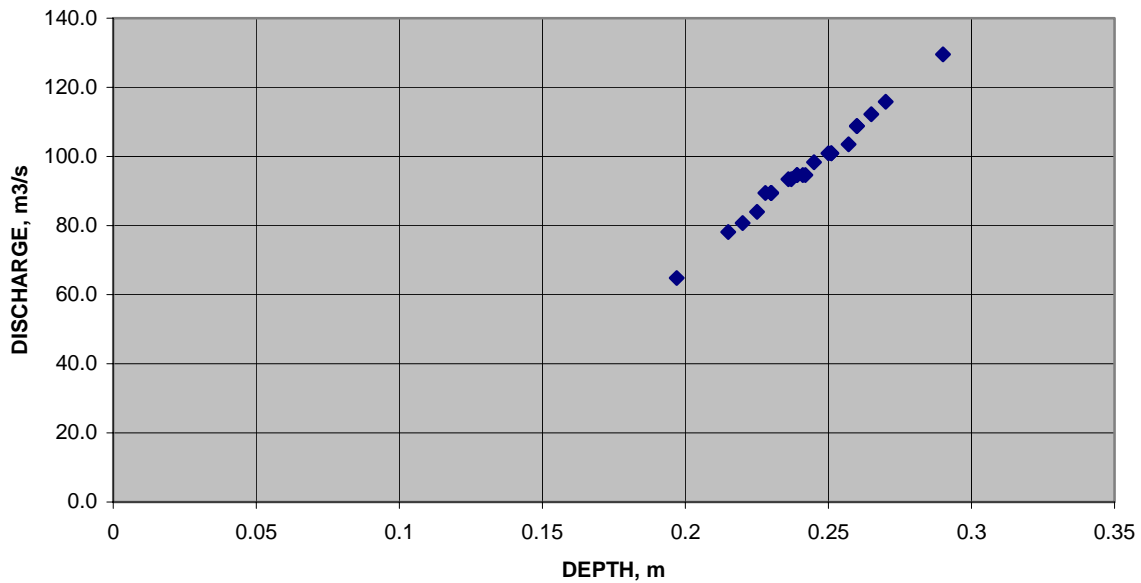


Figure 4.7(b): Depth-discharge plot, J5 - 10/06/04



These two plots show a near “concave-upward” pattern that is normally expected for the depth-discharge relationship. The high night flow component is a possible reason for this pattern not being fully depicted, as the range between day and night flows is quite close. (See plots 4.4 (a – d))

#### 4.6.2 Storm water drainage system measurements

The findings on the wastewater flows generated in Alexandra discharged of through the storm water drainage system are summarized in table 4.2 below.

Table 4.2: Summary of flow measurements at the outlets of storm water culverts

Measurement location	Total flows measured from storm water culverts in l/s	
	Max. ADF	ADF
Manhole along diversion from northern and middle storm water culverts	105.32	70.87
Manhole along diversion from southern storm water culvert	311.1	225.3

These measurements were taken during the dry weather period only and thus are assumed to exclude the storm water runoff component. The depth measurements could only be done

during the times when the grids at the outfall end of drains were cleaned out, as blockage from litter trapped in slots caused the wastewater to over flow directly into the Jukskei.

The flows in storm drains are also considered to represent wastewater generated by residents with limited access to ablution facilities particularly the shack dwellers illegally occupying the tributaries of the Jukskei River. The charts in figure 4.5 and 4.6 below are showing the average flow rates measured as derived from depth of flow measurements taken at these manholes using the theory of pipes flowing partially full. The details on calculations of flows are included in appendix C.

**Figure 4.8(a): Flow measurements along pipeline diversions from north and middle storm water culverts (undertaken in April 2004 as part of data collection for research study)**

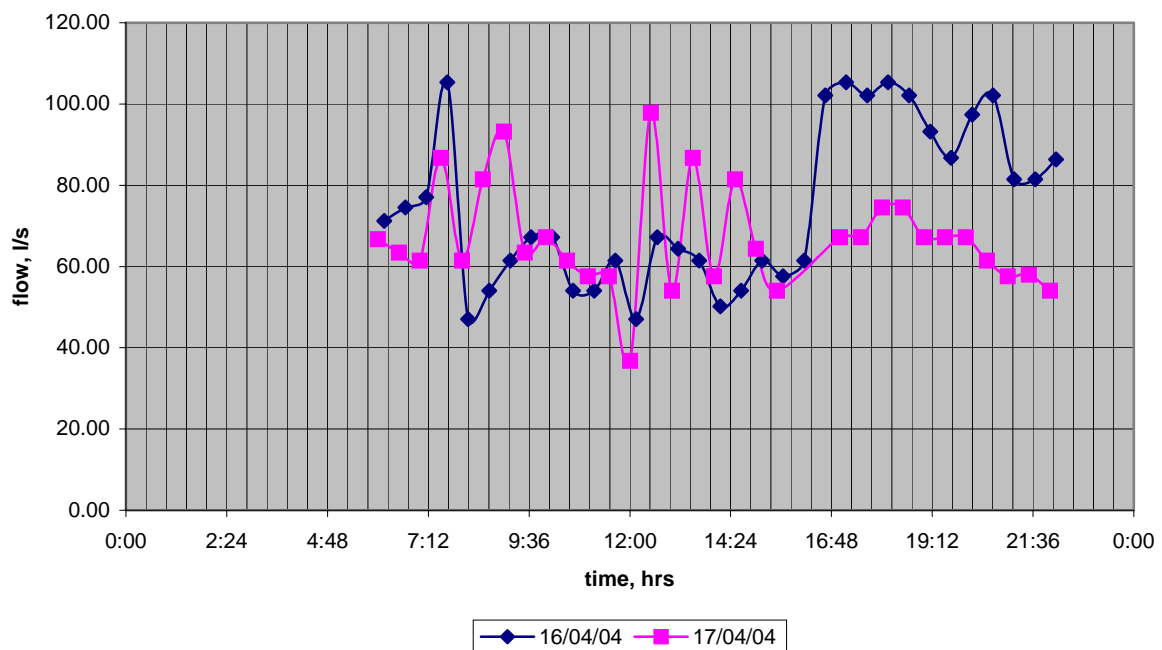
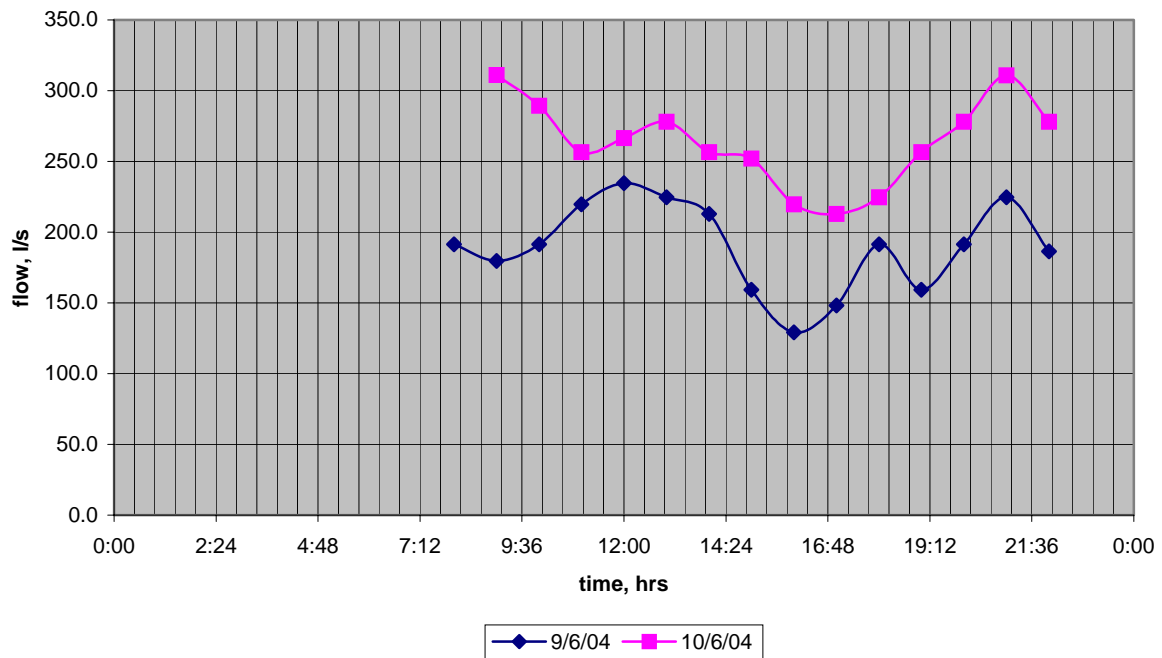




Figure 4.8(b): Flow measurements along pipeline diversion from southern culvert  
(undertaken in June 2004, as part of data collection for research study)



#### 4.6.2.1 Accuracy of flow measurements at outfall of storm drainage channels

The discussion on accuracy of flow data in *section 4.6.2.1*, applies under this section and thus is not repeated, however the figures 4.9 (a) and (b) of the depth-discharge relationships for flow taken at the storm drainage outfall are shown below.

Figure 4.9(a): Depth-discharge plot for north and middle culvert - 16/04/2004

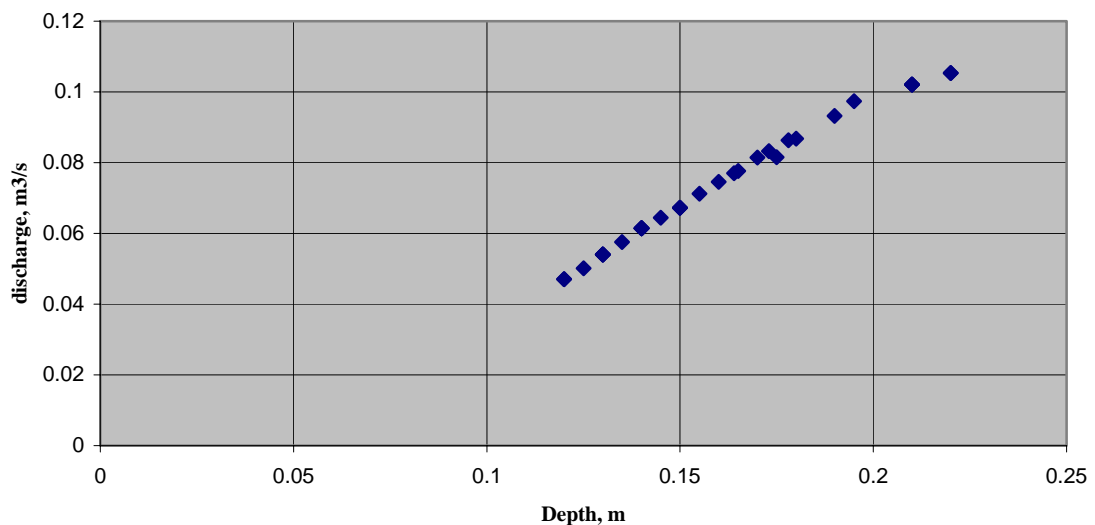
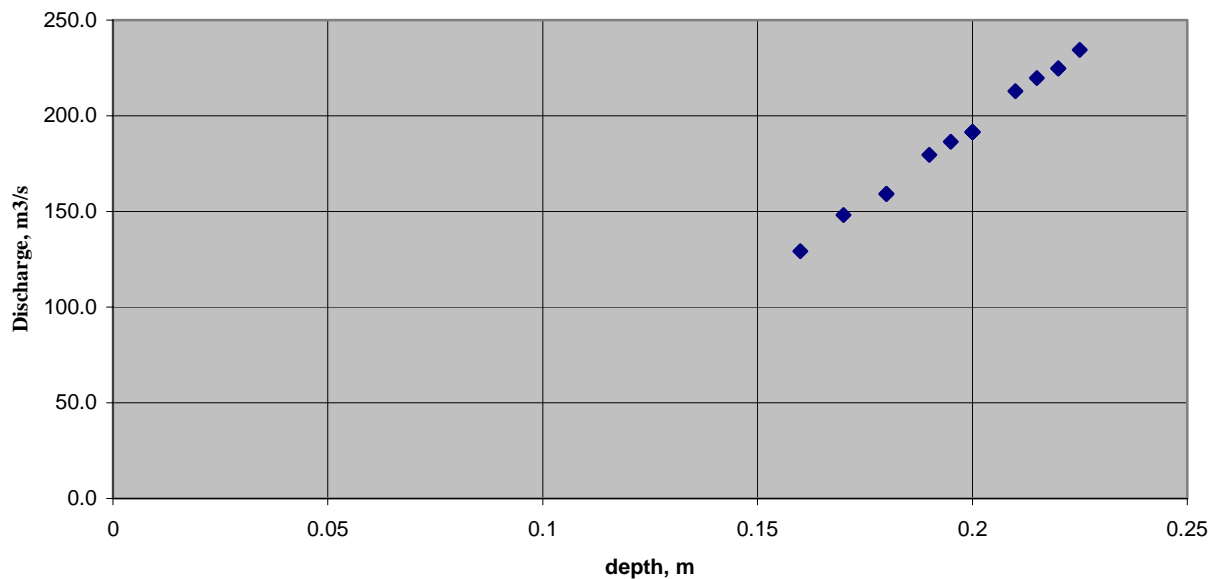


Figure 4.9 (b) Depth-discharge plot, S-CULVERT - 9/06/04



The depth-discharge plots above also show a near “concave-upward” pattern, with the points fitting closely to what would be the straight section of the curve. This was therefore interpreted as fairly reliable data.

#### General discussion on results in relation to demographics

It has not been possible to obtain records on the actual number of water supply and sewerage connections in Alexandra west bank township as well as detailed demographic information on the actual number of households, thus making it difficult to accurately compute the average per capita flows from the total wastewater generated. However, the ARP team has provided a schedule (details attached in appendix B) of erf density, residential structure density and area coverage for approximately 80% of case study area from which a crude estimate of per capita flows will be computed for the entire area, and also the two sewer sub-catchments A, and C, D, E & F.

Table 4.3: General population and demographic information on Alexandra (GMKS, 2000; ARP, 2000; PH&R, 2003; Questionnaire, 2003)

Erf/density i.e. erf/ha	28
Residential structure density, residential units/ha	227.18
Household population (estimate)	4.6
Population per stand	46
Estimated population	350,000-370,000
Areal extent, ha	350

In the table below the population is calculated using both the erf density and residential density derived from ARP demographic data and the values obtained are compared with the generally accepted estimate.

Table 4.4: Summary of population estimates derived from ARP records on erf and residential density information

<b>Erf density</b>	<b>No. of erfs</b>	<b>Population per erf</b>	<b>Total population</b>
28	9,800	46	450,800
<b>Residential density</b>	<b>No. of residential units</b>	<b>Population per unit</b>	<b>Total population</b>
227.18	79,513	4.6	365,760

The residential density gives a value of the population estimate that correlates well with the estimate from other sources, thus, it is used in the computations of per capita flows from the two major sewer sub-catchments as well as the entire Alexandra west bank.

Table 4.5: Estimates of per capita wastewater flow generation basing on June flow measurements

<b>Sewer sub-catchments</b>	<b>Area, ha</b>	<b>No. of residential units (based on 227.18/ha)</b>	<b>No. of people, (based on 4.6 persons/hhold)</b>	<b>Average daily flows monitored, l/s</b>	<b>Per capita flows l/c/d</b>
<b>A &amp; B</b>	175.2	39,802	183,089	487	229.82
<b>C, D, E, F &amp; G</b>	161.3	36,644	168,563	161.97	83.0
<b>ALL</b>	350	79,513	365760	648.97	153.3

## NOTES:

- 1) The flows from storm drains and sewer collector mains were summed in computing per capita flows. The flows from south storm drain added to those from collector mains serving the sewer sub-catchment A while the flows from north and middle culvert added to those measured from collector main serving C, D, E & F. The number of sewerage connections not known and thus the need for this addition in computing per capita flows from the particular section being considered.
- 2) There were no direct measurements of flow taken from collector mains serving sub-catchments B and G, however, in computing per capita flows, their population contribution was included.
- 3) The sewer sub-catchment A is the part of Alexandra west bank that is most formalized (i.e. formal houses, flats/apartments and institutions), and this could be the reason for the higher sewage flows.
- 4) The sewer sub catchments C, D, E, F and G are in the part of Alexandra that is more densely occupied by shacks with limited accessibility to water supply and sewerage (i.e. shared facilities) and thus the probable reason for low sewage generation.
- 5) The per capita flows from sewer sub-catchment A with more formalized stands is approximately three times that of the section of Alexandra with more informal occupation represented by the sewer sub-catchments, C, D, E, F and G. This is also evident in the sewage flow measurements undertaken by PH&R, (2003).

### **4.6.3 Observations from household water use and wastewater disposal monitoring**

#### Water consumption patterns

The actual consumption by each household as observed throughout a selected day is presented in figures 4.10(a)-(c) below, with the amounts per specific purpose during a given time period. Since records were taken by observation rather than continuous logging using electronic or other equipment, a histogram is used to represent consumption patterns.

Figure 4.10 (a) Household 1: Water consumption pattern on a Tuesday in April 2004

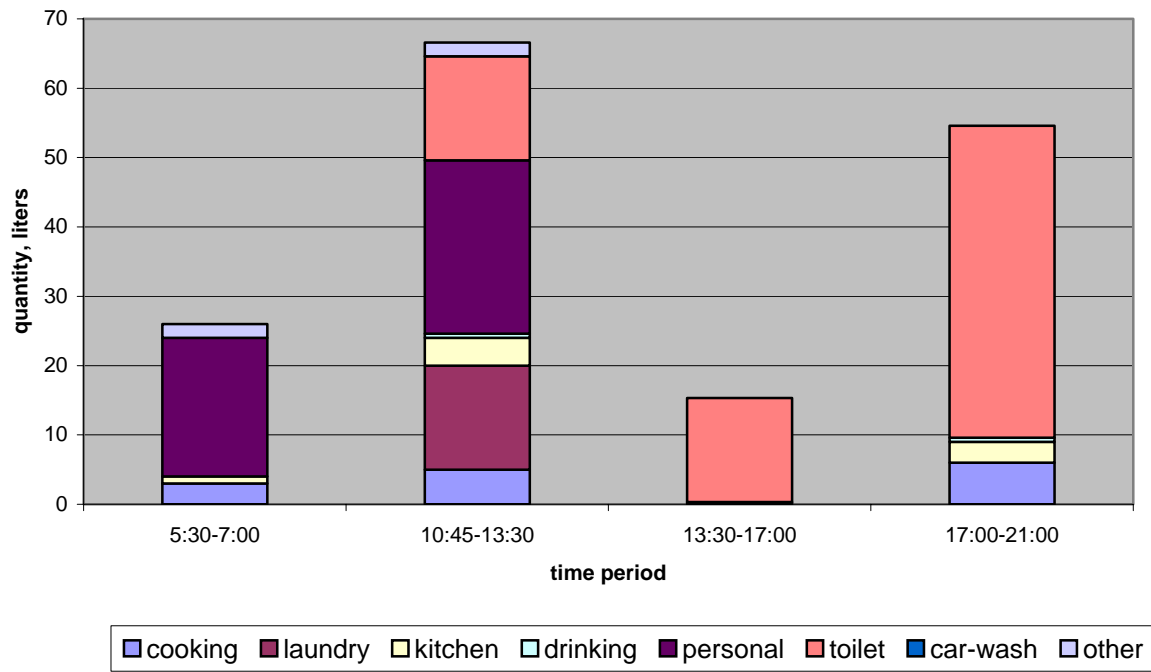


Figure 4.10 (b) Household 2: Water consumption pattern on a Saturday in April 2004

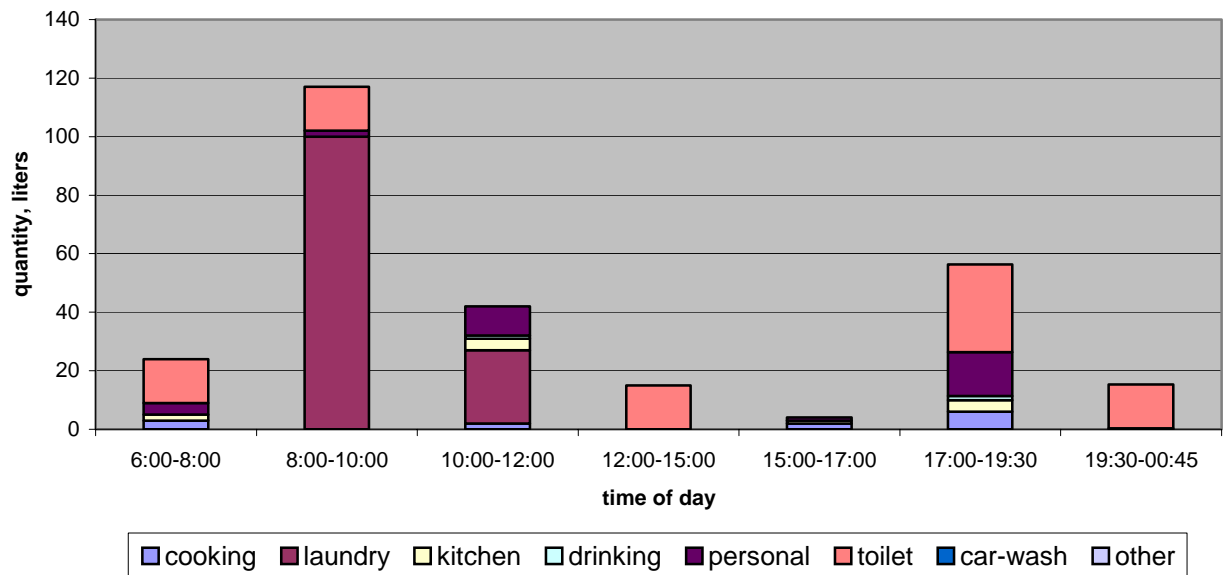
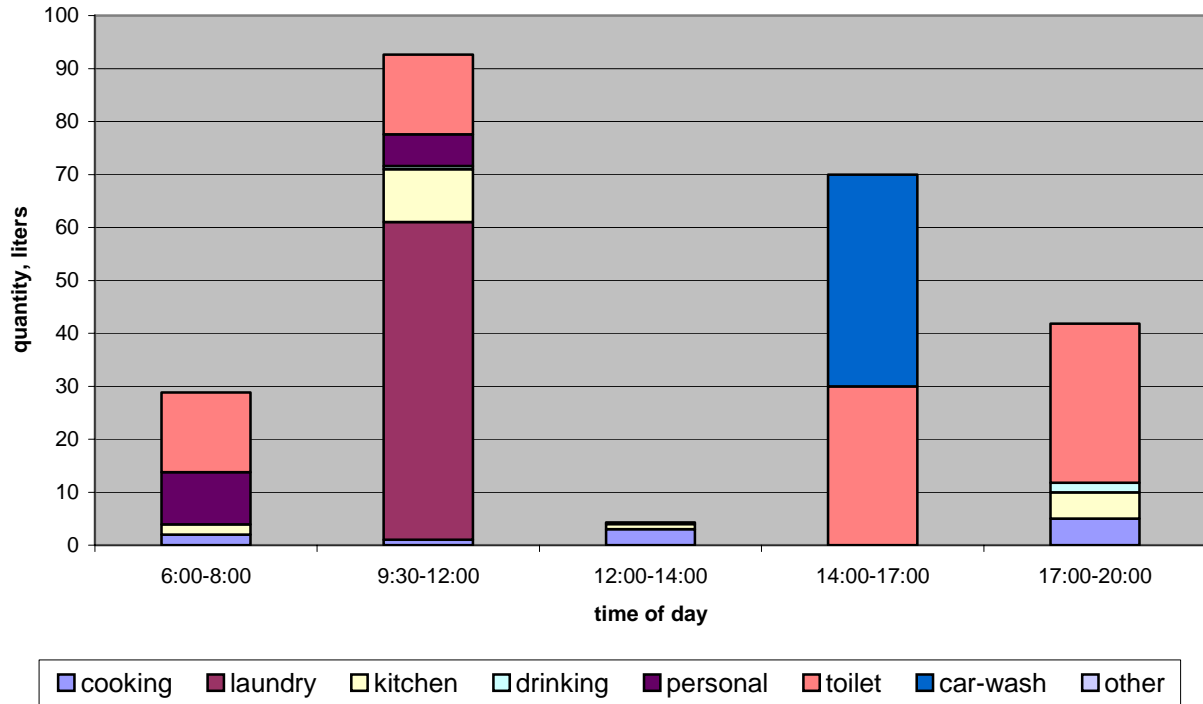


Figure 4.10 (c) household 3: water consumption pattern on a Monday in April 2004



The water consumption pattern appears to be constant throughout the day with high consumption noted in the early morning when the working population is preparing to leave for work, mid-morning when the stay-at-home population is getting active, and the late afternoon to early evening hours when the working population is back from work.

The water consumption rate falls from about lunchtime through to the mid-afternoon times of the day except for the occasional toilet flushing, car-washing and/or clothes washing as seen for household 3 in figure 4.10(c)

#### Specific water use data

The total amounts of water used for the week of observation by the occupants of the three shack dwellings in the backyard of original formal house stand are summarized in table 4.6 below. The per capita consumption per household is also computed and included in table.

Table 4.6: Summary of field monitored individual household water supply consumption

<b>Time period of Observations</b>	<b>Calculated components from measured data</b>	<b>Household 1 (7 persons)</b>	<b>Household 2 (3 persons)</b>	<b>Household 3 (4 persons)</b>
19/04/04 to 24/04/04	Total weekly water consumption, liters/household	883.2	892.6	928.3
	Average observed daily water consumption, l/household/day	147.2	148.8	185.3
	Average observed daily per capita water consumption, l/c/d	24.5	49.6	46.3

*Note:*

- *Household 1 had six occupants, all male and they entertained a visitor at the weekend, household 2, had three occupants, a couple and their child, while household 3 had 4 occupants, all female.*

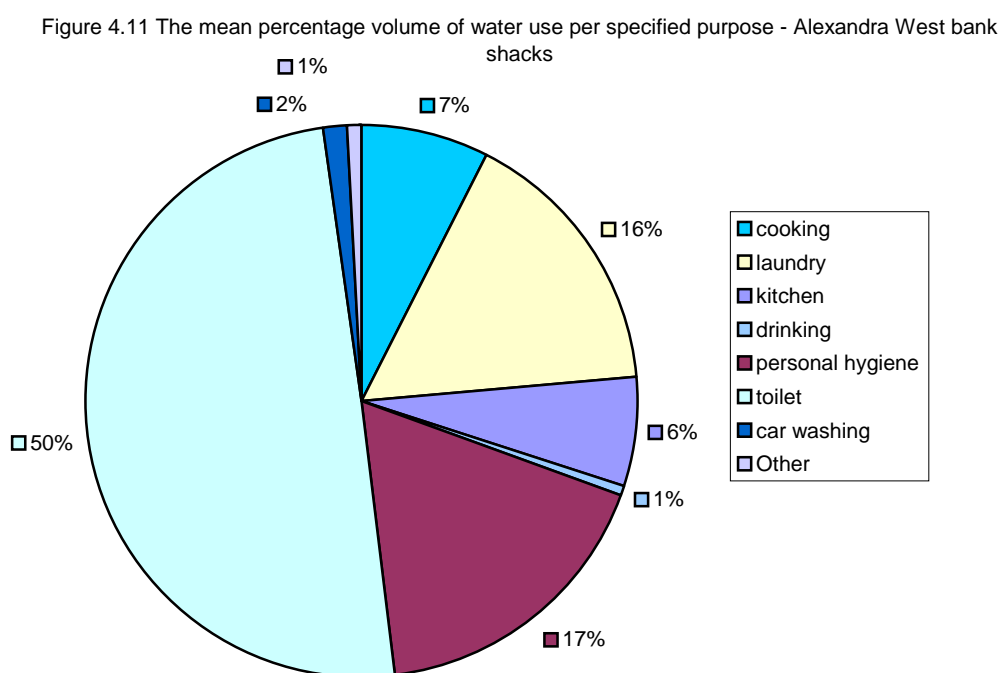
The estimated per capita consumption from the household observations is 40l/c/d, which is 0.57 times that assumed for the area basing on the daily amounts supplied as taken from readings recorded at the three bulk water meters monitored by GMKS. (GMKS, 2000)

The bulk meters were reported as malfunctioning, which could be one reason for this difference, other reasons include the following:

- Leakages from faulty taps and other plumbing fixtures installed in both the communal ablution blocks and in the individual households with a direct connection to the water supply network.
- Wastage by the consumers particularly those served by communal ablution blocks and illegally connected stand alone pipes as they often leave the taps running even when not in use. (See photographs – Appendix A)
- System losses arising from leakages in the pipeline joints and fittings within the reticulation network.
- It is also possible that since only backyard shack dweller category of households were monitored, the consumption is a reflection of only those consumers supplied at this service level. The consumers in formal houses, flats and hostels are likely to have a higher per capita consumption, as is normally the case for individual household

connection. When the per capita consumption is considered collectively, that is even with the inclusion of what it would be for the consumers served by other service levels, it is higher.

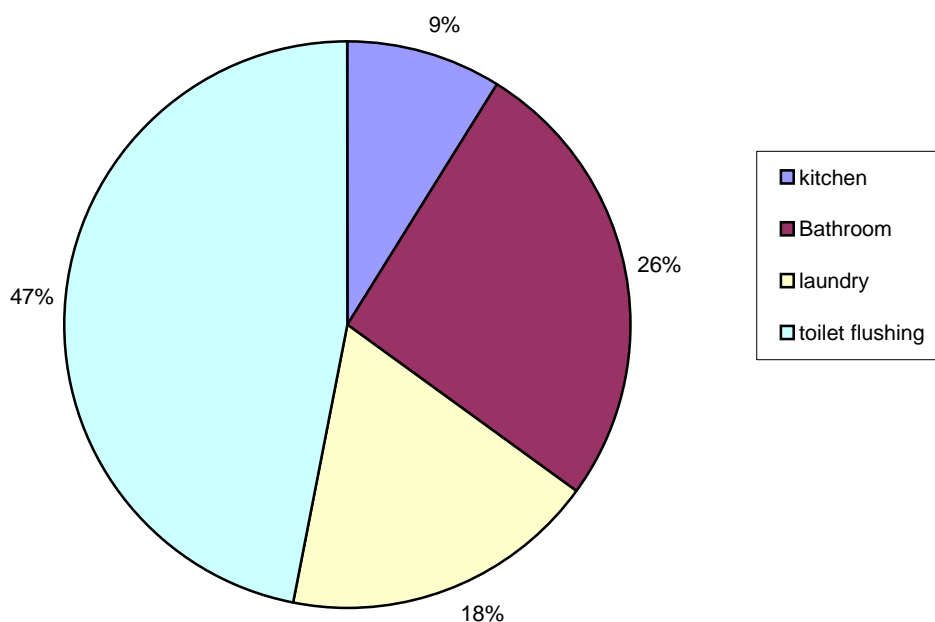
The specific uses of water were similar for all the households monitored and the mean percentage volume consumed was calculated per given purpose for each household and is presented in figure 4.11 below.



The mean percentage volume use for a western style household is presented in figure 4.12 below for comparison.



**Figure 4.12 Mean %age volume of water used per specific purpose - Western style household (Alcock, 2002)**



The toilet, bathroom and laundry uses by shack dwellers and western style households are almost by equal amounts, despite differences in income status, this can be attributed to the fact that water services not being paid for in Alexandra and thus the high usage.

The mean percentage per capita consumption as per the specified purposes for the Alexandra shack household type has also been computed for comparison with the mean percentage per capita consumption for White South African households and rural households in Lesotho, and all three pie-charts are presented in figures 4.13 (a)-(c) below.

Figure 4.13 (a): Mean percentage daily per capita consumption - Alexandra west bank shacks

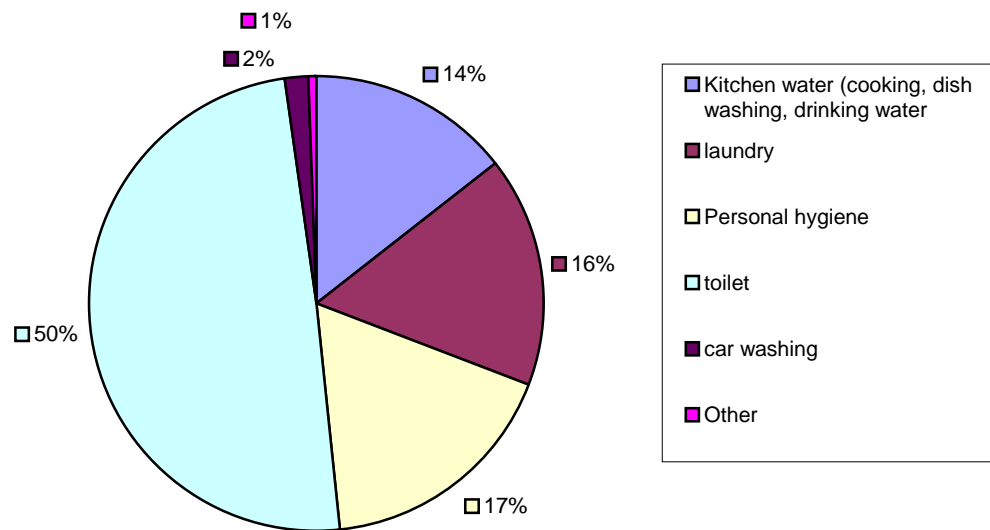
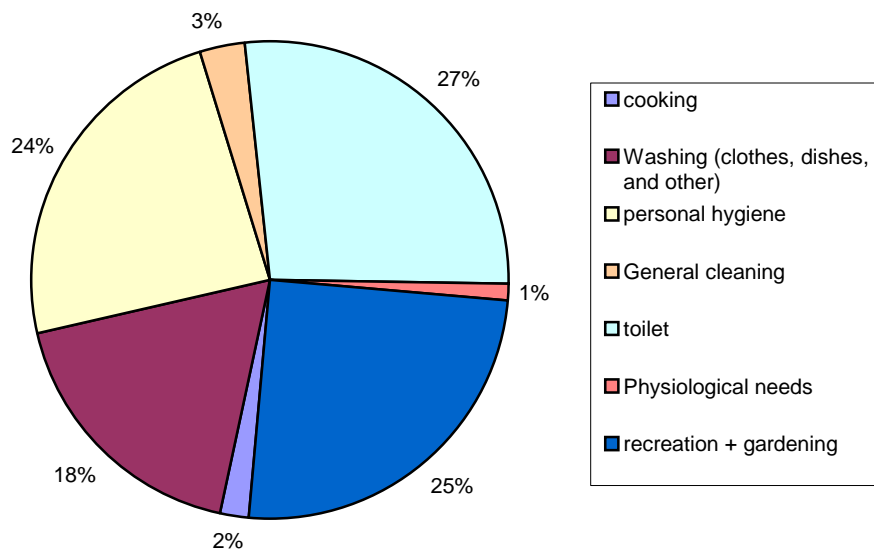
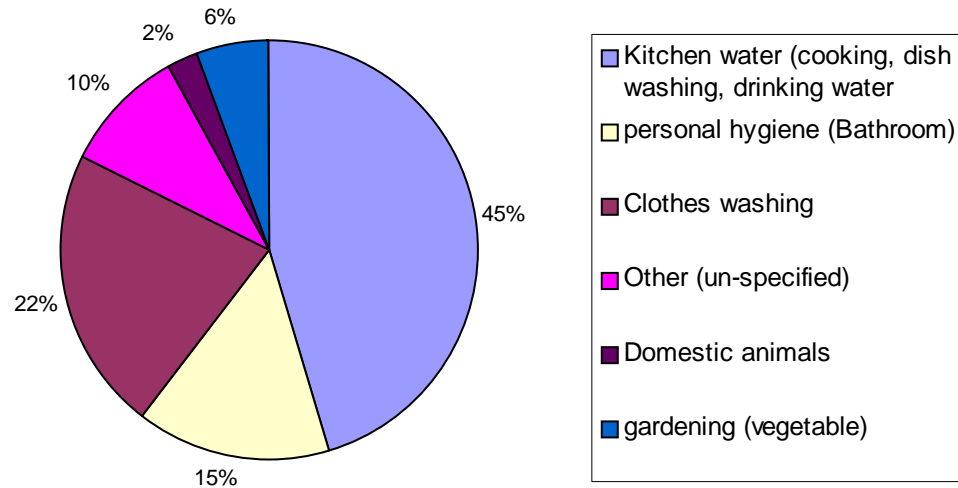


Figure 4.13 (b): Percentage mean daily per capita consumption for specified purposes-White South African households



**Figure 4.13 (c): Mean percentage daily per capita consumption per specified purpose - rural households in Lesotho**



Comparison is limited by the fact that the water consumption surveys carried out for each case grouped the specific purposes of water use differently. In addition water supply service levels also differ, the rural households in Lesotho have distant sources of water supply, while both the Alexandra shack households and White South African households have reticulated supply i.e. close proximity to source of supply. The following has been noted however, for those water use purposes that can be compared:

- Toilet flushing water use much higher in Alexandra than in White South African households probably because of the stay home population in this settlement. The rural households of Lesotho use pit latrines.
- The water for personal hygiene purposes appears to fall within the same range for all three household types.
- Kitchen water use high for rural households high probably because, residents have all meals at home, while urban and peri-urban residents may opt to eat out for the high-income earners, while the stay at home population in Alexandra chooses a light meal.

### Wastewater flows

The wastewater flows generated by the shacks monitored were computed by assuming that apart from the water used in cooking and drinking, the balance of the household water consumption constitutes the wastewater. In table 4.7 below, is the summary of wastewater flows per household on a weekly, mean daily and mean per capita basis.

Table 4.7: Summary of field monitored individual household wastewater disposal characteristics – Alexandra west bank backyard shacks

<b>Time period of measurement</b>	<b>Calculated components from measured data</b>	<b>Household 1 (7persons)</b>	<b>Household 2 (3 persons)</b>	<b>Household 3 (4 persons)</b>
19/04/04 to 24/04/04	Weekly wastewater disposal, liters/household	791	754.4	868.5
	Weekly waste water disposal as %age	89.6	84.5	93.8
	Average observed daily wastewater disposal, l/household/day	131.8	125.7	173.7
	Average observed daily wastewater disposal per household, as %age	89.6	84.5	93.8
	Average observed daily per capita wastewater disposal, l/c/d	21.9	41.9	43.4
	Average observed daily per capita wastewater disposal as %age	89.6	84.5	93.8

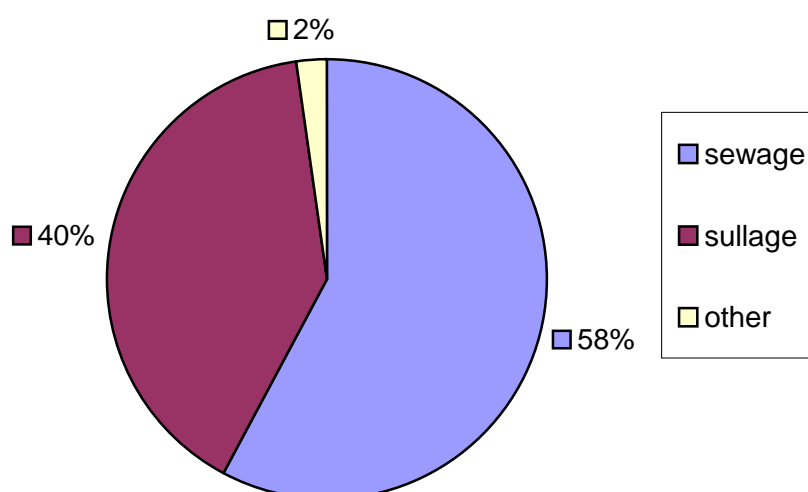
From the table above it is seen that the proportion of water used that reaches the sewerage collection systems also known as the return factor ranges from 0.85-0.94. This range is arrived at by subtracting the amount of water used for drinking and cooking purposes from the daily household consumption. The remaining amount is then expressed as a fraction of the total consumption/household/day and an average obtained for each household over the

entire week. In order to separate the grey and black water waste streams, the following assumptions are made:

- The wastewater generated from laundry, kitchen, and personal hygiene constitute the “grey” water.
- Black water comprises wastewater generated from toilet flushing,
- The component termed other refers to the water used in car washing and that used to wet the ground so as to control dust pollution.

In figure 4.14 below, the mean percentage volume of each component waste streams is presented in a pie-chart format.

Figure 4.14: Mean percentage volume of selected waste streams of the total sanitary wastewater generated at household level



In table 4.8 below, the findings of the sewage flow estimation section of this report are summarized.

TABLE 4.8: Summary of findings on wastewater flow generation from study area

Approximate return factor for Alexandra west bank	<b>0.9</b>
Average per capita wastewater flow rates, l/c/d	
General	<b>153.3</b>
Formalized section	<b>229.8</b>
Informal section	<b>83</b>
Specific to residents supplied at intermediate service level	<b>35</b>
Total flows, m <sup>3</sup> /ha/day	<b>160.2</b>

The flows generated in  $\text{m}^3/\text{ha}/\text{day}$  exceed the  $10\text{m}^3/\text{ha}/\text{day}$  limit for on-site management, and as concluded in chapter three off site sewerage is necessary. The water supply consumption levels observed from household monitoring, which are likely to be the minimum amounts for the area, are adequate for the requirements specified for the installation of simplified (shallow) sewerage systems.

## **PART B: STORM WATER RUNOFF ESTIMATION**

### **4.7 AVAILABLE METHODS**

The principles used in estimating quantities of storm water runoff from urban drainage areas and other small watersheds have progressed from simple rules of thumb to complex simulation methods that incorporate fundamental hydrological processes, (Viessman, Knapp & Harbaugh, 1977). The approaches that embody these principles are described here below.

#### **4.7.1 Simple rules-of-thumb**

These were early guidelines on the amount of rainfall that would contribute to the runoff for a particular storm event. For the English, the assumption was that about half of the rainfall would appear as runoff from urban surfaces.

#### **4.7.2 Macroscopic approach**

These second generation approaches are crude empirical formulas particularly the Rational method and the Unit Hydrograph method, that consist of the following procedures:

- Consideration of the entire drainage area as a single unit
- Estimation of flow at only the most downstream point
- An assumption that rainfall is uniformly distributed over the drainage area

The **RATIONAL METHOD** is described by the statement,  $Q = CIA$ .  $Q$  equals the peak runoff rate,  $C$  is a runoff coefficient (the ratio of an instantaneous peak runoff rate to a rainfall rate averaged over a time of concentration,  $I$  is the rainfall rate,  $A$  is the size of the drainage area, all in compatible units.

The rationale for the method lies in the concept that application of a steady, uniform rainfall intensity will cause runoff to reach its maximum rate when all parts of the watershed are contributing to the outflow at the point of design. This condition is met after the elapsed time,  $t_c$ , the time of concentration, which is usually taken as the time for water to flow from the most remote part of the watershed. (Viessman, Knapp & Harbaugh, 1977)

The concept of the **UNIT HYDROGRAPH METHOD**, involves construction of the hydrograph for a storm of any magnitude and duration from a unit hydrograph. The unit

hydrograph is the hydrograph of 1 mm of runoff from a drainage area produced by a uniform rainfall lasting any unit of time. (Viessman, Knapp & Harbaugh, 1977) This method is based on the assumption that hydrographs can be added linearly.

Other methods include: (Stephenson, 1981)

The **STEP METHOD**, which overcomes a major shortcoming of the rational method by accounting for the flow time through each drain in the accumulation of flow and thus allows for computation of drain sizes.

The **TIME AREA DIAGRAM AND ISOCHRONAL METHODS**, which assume that every point in the catchment has a unique travel time to the mouth or point of discharge. The points of equal travel time are joined together forming a line of constant travel time called an isochrone, and in this way the drainage area under study is demarcated into time zones. A time-area diagram showing the rate of build-up in contributing area during the storm can then be plotted, yielding a mass area curve, the slope of which is proportional to the flow rate.

The **TANGENT METHODS**, these relate the time-area graph developed in the above method to the storm intensity-duration relationship. (Stephenson, 1981)

#### **4.7.3 The microscopic approach**

This approach involves the computer simulation of urban runoff characterized by an attempt to quantify all pertinent physical phenomena from the input (rainfall) and output (runoff). (Viessman, Knapp & Harbaugh, 1977) It is the result of advances in computer technology and the inadequacy of the manual and graphical approaches described above in providing an accurate assessment of catchment responses to inform modern storm water management policy in addressing issues like effect of urbanization on the hydrological regime. (Green, 1984)

A multitude of hydrological simulation models differing widely in applicability as well as complexity are available. Generally they take into account most if not all the fundamental hydrological processes involved, provided the equations governing these processes are known. They are also able to give an output at points of interest in time and space, (Green, 1984). Some of these models are oriented towards single events whereas others perform continuous simulation. (Stephenson, 1981) The following urban water shed models have been tested for application in the South African situation.

- **STORM WATER MANAGEMENT MODEL** (Green, 1984)

This model is capable of simulating runoff quantity and quality, dry-weather flow, treatment facilities and receiving water quality. It can be operated in a single event or continuous

simulation mode. The flow routing procedure is based on numerical solution of the kinematic equations. It is specifically tailored to urban drainage application

- **ILLUDAS-SA** (Green, 1984)

This isochronal model utilizing the time-area runoff routing method is a modified and metricated version of the Illinois Urban Drainage Area Simulator, (ILLUDAS). It is most useful in a design situation.

- **KINE 2** (Green, 1984)

This model employs two-dimensional kinematic equations for flow routing and can be used in design for the prediction of runoff hydrographs as well as for assessing the effect of man-made changes on runoff. It is ideally suited to small rural catchments, as it is inappropriate for modeling areas where a pipe or channel system significantly affects flow characteristics at the outlet.

- **URBCEL**

A single event simulation model best suited to larger urbanized catchments i.e. drainage area  $> 10\text{km}^2$ .

- **WITWAT** (Green, 1984)

This model employs the numerical solution of one-dimensional kinematic equations for overland flow routing. It routes flows through the conduit network using either time-shift routing or kinematic routing. It is useful in designing pipe sizes within a network, and can also output the hydrograph at a specified pipe node depending on the mode of application of the model.

- **WITSKM** (Wimberley, 1992)

This model adopts a modular approach to model the catchment response to a time-series of rainfall. Flow over pervious and impervious surfaces, and in pipes and channels is modeled using the kinematic routing approach, notably the Muskingum-Cunge method. Infiltration is simulated using the Green-Ampt model. It was originally developed to simulate single events, but has subsequently been enhanced to provide continuous simulation.

- **RAFLS** (Nsibirwa, 2003)

This is a single event model written in the visual basic programming language with a graphical interface for input of the necessary data for simulation of a synthetic or actual hydrograph for a given storm event. It was developed at the University of the Witwatersrand, by Stephenson and Randell. It incorporates the kinematic theory and takes into account runoff as both unsteady and non-uniform.



## **4.8 DATA REQUIREMENTS AND AVAILABILITY**

The key data requirements for storm water runoff estimation regardless of the method used are:

- Physical catchment characteristics i.e. areal extent, slopes, ground cover, land use types, soils and geological structure. They can be reliably obtained from studying areal photographs, topographic and geology maps of the watershed.
- Hydrologic data obtained through examination of rainfall, storm pattern and runoff records of the watershed being studied.

The physical data can be estimated from the sources specified and thus is generally available. In the case of hydrologic data, instrumentation of urban catchments in South Africa for the purpose of rainfall and runoff data measurement has not been extensively carried out. However, the depth-duration-frequency relationship for point rainfall in South Africa has been determined (Midgley and Pitman, 1978) and a co-axial diagram derived from statistical analyses of previous rainfall temporal and spatial distribution, it is reproduced in appendix D. In addition, Smithers and Schulze, (2002) have more recently developed a computer program to estimate design rainfall depths for any location in South Africa. These sources are sufficient to provide the necessary hydrologic data for storm water runoff estimation from the study area.

## **4.9 ADOPTED METHODS AND JUSTIFICATION**

The advances in computer technology that have led to proliferation of hydrological computer models which give more accurate results with less effort, make the use of empirical methods unnecessary. In addition the computer simulation models have the added advantage of being able to synthesize hydrographs from which runoff volumes can be estimated. For purposes of this research the RAFLS single event models is to be utilized in the estimation of storm water flows from Alexandra west bank.

## **4.10 RAFLS**

### **4.10.1 System configuration**

The RAFLS program simulates flow for a system that can have up to four distinct module types namely catchments denoted by number 1, conduits by 2, channels by 3 and reservoirs by 4. A total of 50 modules can be entered for one simulation.

The **numbering system** specified requires that catchment type modules be listed before the conduits or channels they feed into, which in turn should be listed before the reservoirs they feed into.

#### **4.10.2 Input data requirements**

- *The schematic layout of the drainage system*, shown in the figure below.
- *Catchment properties*, RAFLS specifies the following terms to represent each sub-catchment's characteristics. It also recommends values or ranges of values as deemed suitable by the models' developers.
  - Aquifer penetration which has to do with initial moisture content
  - Rill ratio, which is the proportion of land occupied by overland flow.
  - Direct runoff percentage
  - Soil vertical and horizontal permeability and porosity of soil which affect infiltration particularly during the latter part of the storm.

The recommended values and ranges of values are indicated beside each term on the input sheet for the model, see appendix D.

- *Physical dimensions of the modules namely*,
  - Mannings roughness coefficients for the modules can be obtained from literature on South African conditions.
  - The length, width, gradients and slopes of the modules and these are directly measurable from topographic maps of the area also specified by the user.
- *Hyetograph data* i.e. the rainfall intensity and critical storm duration for a given frequency level.

### **4.11 RAFLS' SIMULATION OF STORM WATER FLOWS IN ALEXANDRA**

#### **4.11.1 Catchment discretisation and system numbering**

The discretisation adopted is dictated by the three existing subsurface culvert. The study area is discretised into five sub-catchments with three of them draining into these culverts while the drainage pattern depicted by the contours for the other two indicate that they drain directly into the Jukskei River. Figure 4.12 below shows the discretisation.

For the system numbering, the existence of the storm pipe network connected to the subsurface culvert pipes is disregarded for simplification, thus flow is assumed to be overland from each sub catchment discharging directly into it's natural drainage path, and the section of culverts also assumed to be a natural drainage path of the same dimensions.

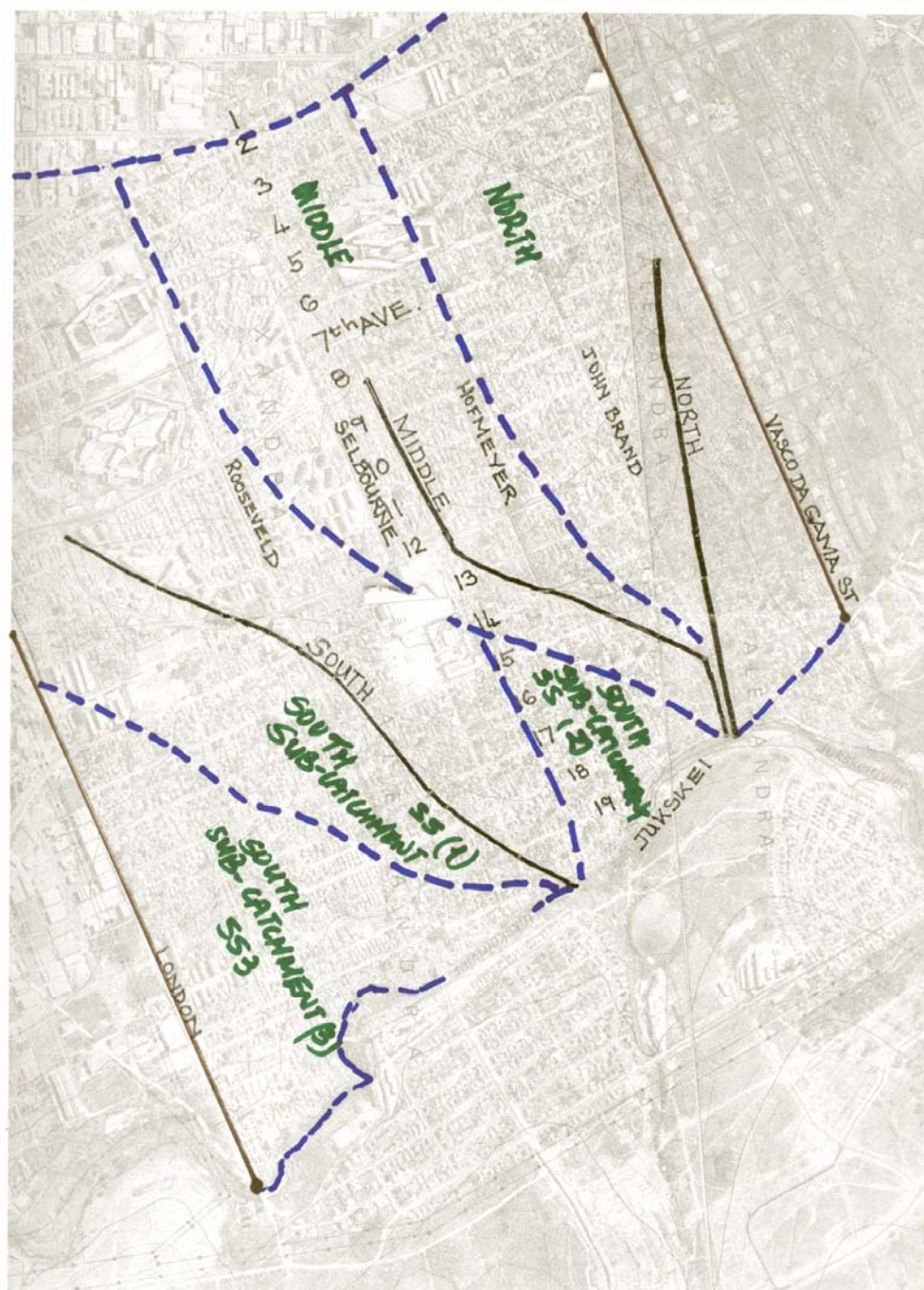


Figure 4.15 Discretisation of Alexandra watershed area

#### 4.11.2 Model input parameter values

##### 4.11.2.1 Modules

For purposes of simplification, only the catchment and channel module types are in use basing on the discretisation adopted.

###### a) Sub-catchments

The length, width, area and gradients for each are taken from a 1:10,000 ortho-photo map of the study area. Their mannings' roughness coefficients are taken from guidelines given by Stephenson and Coleman (1990). The values for the other catchment properties are taken as per the proposed values and ranges of values for the model.

###### b) The channel

The length of the natural drainage path, is the same as for the sub-catchment drained and it is as determined from the ortho-photo map, the width, however, is assumed to be the same as that of the subsurface culvert. The Mannings' roughness coefficients also obtained from Stephenson and Coleman, (1990) guidelines while side slopes are assumed to be negligible.

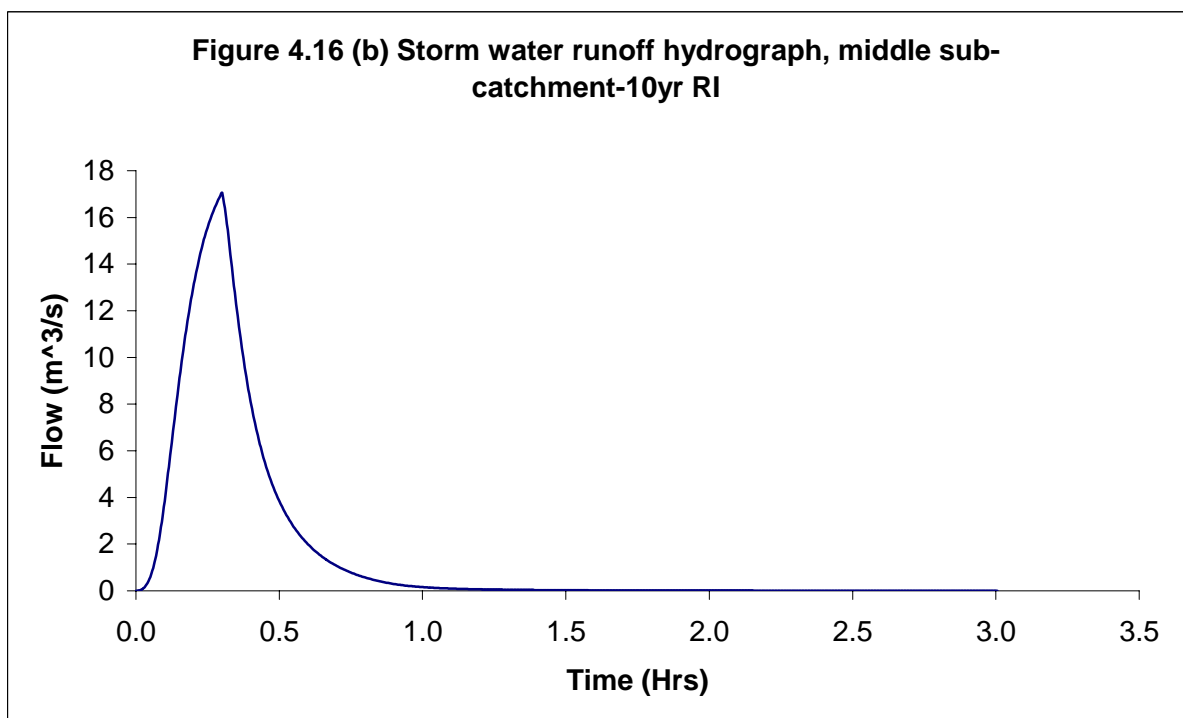
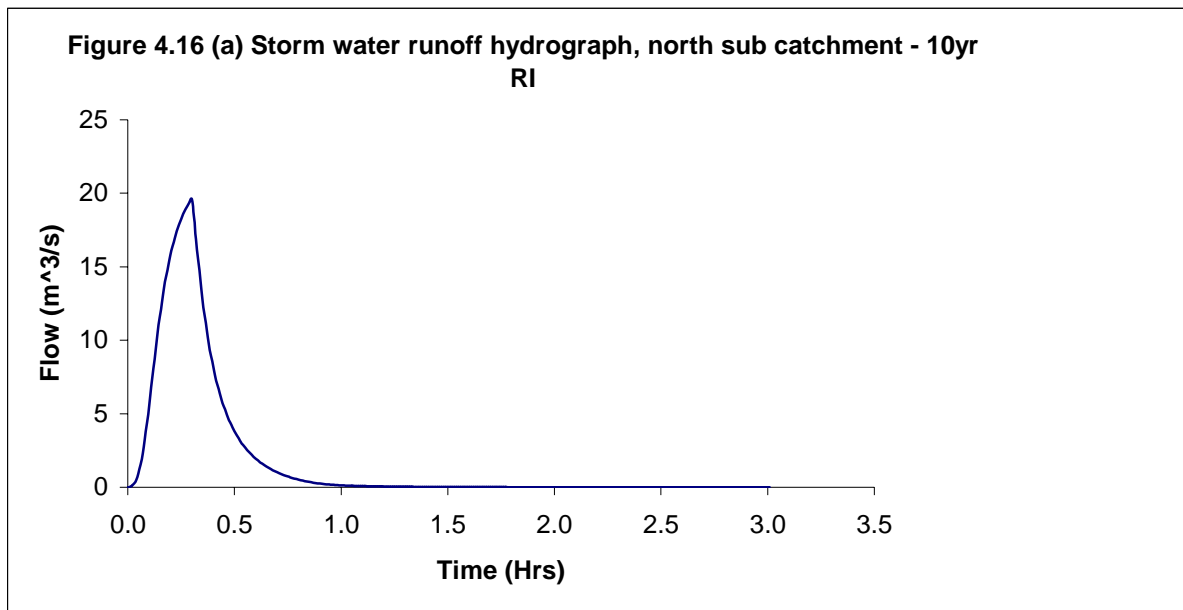
##### 4.11.2.2 Hyetograph

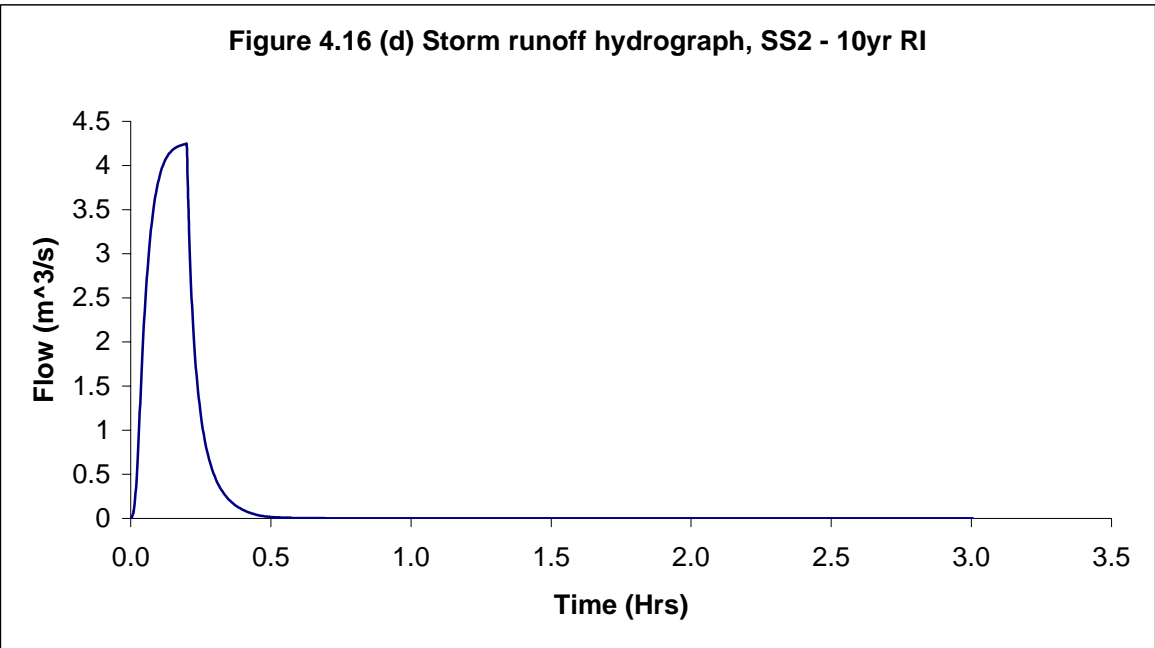
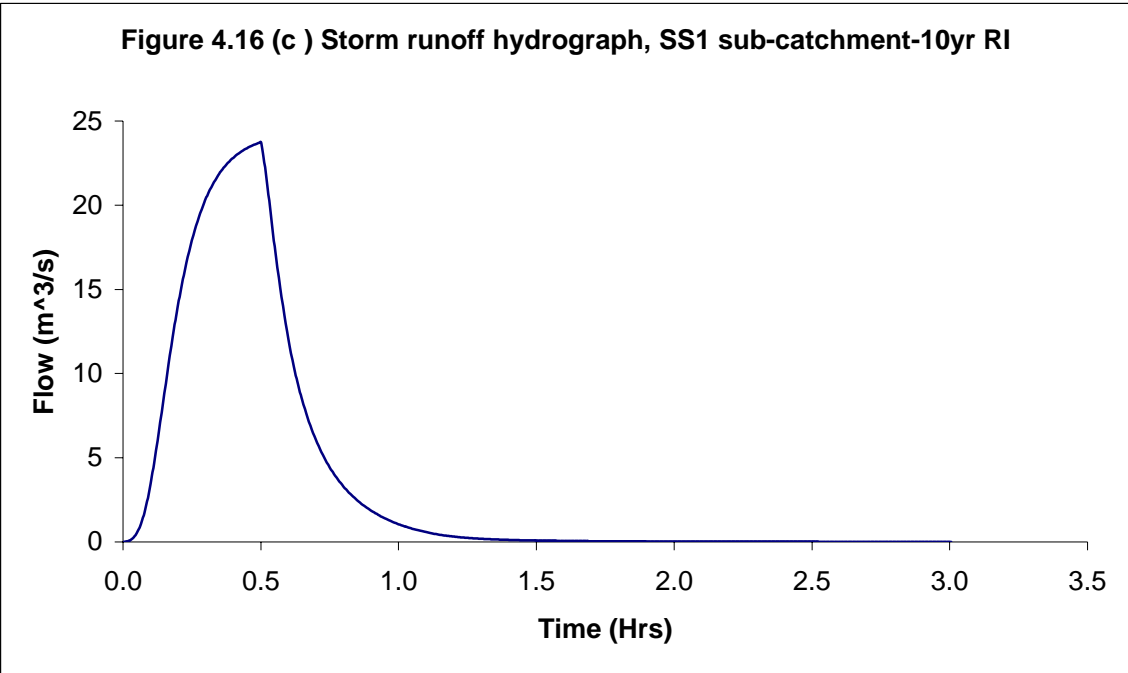
The input rainfall data is abstracted from the co-axial graph for depth-duration-frequency relationship for point rainfall in South Africa by determining the critical duration for the catchment i.e.  $t_c$  and thereafter determining storm depth basing on the Mean Annual Precipitation (MAP) for Alexandra which is 750mm taken from an isohyetal map of Johannesburg (Wilcocks, 1970). This is done for the 10-yr frequency level only, since runoff flows from higher frequency levels, normally to high, it is not feasible for them to be transported through pipe networks. The rainfall intensity values obtained for each of the sub-catchments are summarized in the table 4.9 below.

Table 4.9: Rainfall intensity values derived from Midgley's co-axial diagram

Sub-catchment	North	Middle	South - 1	South - 2	South - 3
$t_c$ , minutes	20	20	30	10	15
Rainfall intensity, mm/hr (10yr Recurrence interval)	96	96	85.7	141	120

In the figures 4.16 (a)-(e) below are some of the rainfall hydrographs generated by simulation of storm water runoff flows from Alexandra west bank using RAFLS.





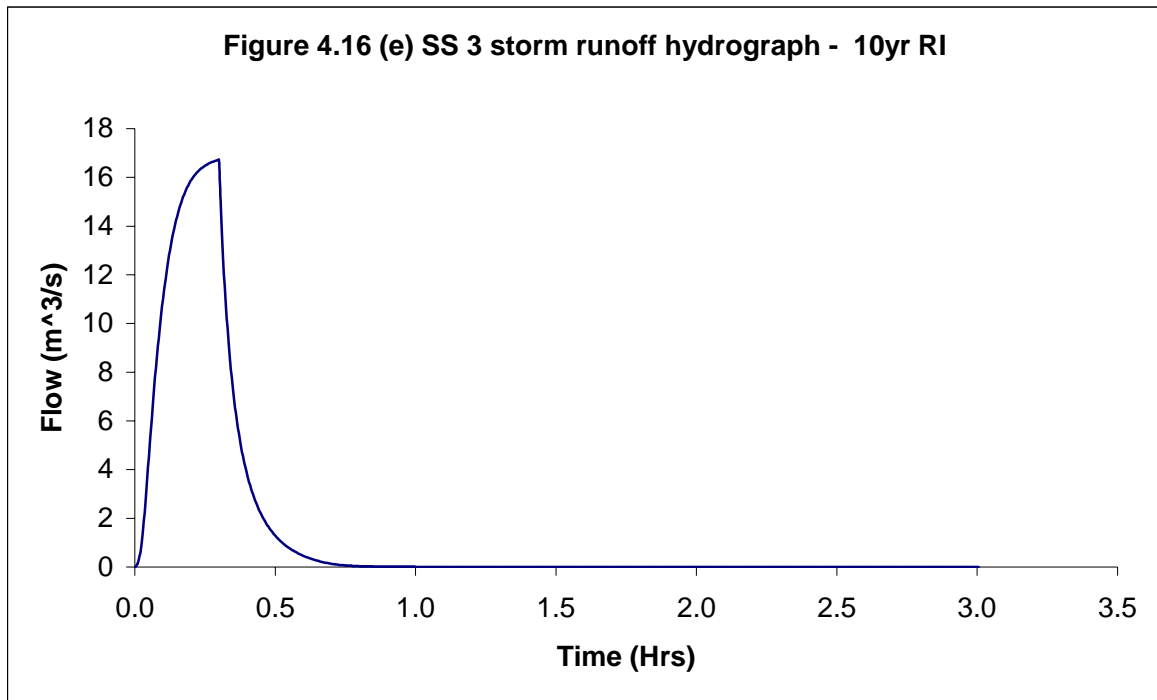


Table 4.10: Peak flows from each sub-catchment for the 10-year recurrence interval

Sub-catchment	North	Middle	South - 1	South -2	South - 3
<b>Area, ha</b>	80.19	89.82	115.64	11.95	55.96
<b>Peak flow, m<sup>3</sup>/s</b>	19.62	17.06	23.74	4.24	16.72

The storm runoff peak flows computed are for the entire drainage areas as per the discretisation shown in figure 4.15. The black and grey water flows generated, however were for only three households on a given stand. An attempt was made therefore to determine storm runoff flows from an area equivalent to the average stand size in Alexandra, which is approximately 0.05ha. Taking a time of concentration equal to 6minutes, a rainfall intensity of 166.67 mm/hr was derived from the Midgley's diagram for the 10-yr level flood. Using the rational method, the storm runoff peak flows computed as follows:

$$\begin{aligned}
 Q_p &= CIA, \text{ with } C \text{ taken as } 0.8, \\
 Q_p &= 0.8 * 166.67 / (1000 * 3600) * 500 \\
 &= 0.0185 \text{ m}^3/\text{s}
 \end{aligned}$$

This flow is much larger than the combined peak flow for the black and grey water flows generated by the estimated 10 households per stand, and thus couldn't meaningfully be represented in a pie-chart format.

#### 4.12 SUMMARY OF DRAINAGE FLOW ESTIMATIONS

The findings on drainage flows generated within the Alexandra west bank area are summarized as follows:

- The minimum production rate for the wastewater flow component of the drainage flows is 35l/c/d, which equates to 37m<sup>3</sup>/ha/day, when computed for the entire settlement basing on the population estimate of 370,000. On-site management of drainage flows not considered safe when flows exceed a 10m<sup>3</sup>/ha/day production rate.
- Field observations of water use and wastewater disposal practices of residents served at an intermediate level of service, (the lowest in Alexandra) show that grey water and black water waste streams make up 41% and 58% respectively, of the total wastewater flows generated for a given household unit. The black water waste stream considered unusually high for low-income settlements, however, this can be attributed to the large proportion of the population that doesn't move out of the area during the day to go to the workplace.
- The proportions of grey and black water in the wastewater component of the drainage flows are assumed to apply even in the case of residents served at higher service levels, because no significant difference in lifestyles noted, particularly in regards to use of water for recreation or aesthetic gardening purposes.
- In the event of a rainfall event, storm runoff flow estimates from an area the size of an average erf/stand in Alexandra are so much larger than the total wastewater flows from the same area. The composite amounts of each of the three waste streams of concern could therefore not be meaningfully represented in a diagrammatic way e.g. using the pie-chart format for the stand size area considered. This shows that storm runoff flows though intermittent, are the largest of all the three waste streams particularly in the case of an intensively developed site where nature's role in retardation, temporary storage and drainage of flows through soils is interfered with since impermeable surfaces have replaced vegetation.

The total drainage flows from the settlement were determined from field measurements at the outfall end for the wastewater component and computer simulation for the storm runoff flows. They are utilized in the proceeding chapter to compare costs for possible off-site drainage system arrangements.



## **CHAPTER FIVE: COMPARISON OF IDENTIFIED OPTIONS FOR MANAGING DRAINAGE FLOWS FROM STUDY AREA**

### **5.1 INTRODUCTION**

The conclusions of chapters three and four of this report indicate that the use of on-site management technology options to control quantities of drainage flows from Alexandra west bank area would not be feasible given the prevailing physical site conditions and the quantity of flow generated for the three waste streams of concern i.e. grey, black and storm water.

The off-site management technology options generally involve the use of pipe drainage systems to direct flows away from the site of generation. In this chapter, the outfall pipe drain sizes for the possible separate and/or combined system arrangements will be determined following design standards applicable in South Africa and then the costs estimated for comparison purposes.

The sewage flows from sewer sub-catchment A & B and the storm water runoff quantities from the south sub-catchments SS1, SS2 and SS3 will be used for the sizing of drains. They constitute the drainage flows from the south section of the study area.

### **5.2 SEPARATE SYSTEMS**

In South Africa, the practice is to separate foul and storm water drainage sewers, with foul sewers carrying both grey and black water. The separate system arrangement discussed here, however, would entail further separation of grey and black water. Its application, however, should be subject to the community's acceptance of non-water borne sewerage and this because upgrading from small bore to large bore sewers to accommodate toilet water and waste is difficult and expensive. (Stephenson, 2001)

The application of this arrangement is also subject to the quantities of toilet water and waste flows generated which in the case of Alexandra south section have already rendered the use of chemical toilets ineffective, the use of pit latrines is also hindered by the high densities and poor soil drainage conditions. The use of this system for drainage of the south catchment is therefore deemed not suitable.

### **5.3 COMBINED SYSTEMS**

In regions seweraged many years ago and where storm runoff is relatively low, wastewaters and storm drainage are transported in the same pipes, (Stephenson, 1981). In this section the

outfall pipe drain capacity is determined for the following combined system arrangements for the southern section of the study area:

- 1) Combined “grey” and storm water drainage system, with separate “black” water drainage system. This form of arrangement makes separation at the downstream end and treatment of the concentrated stream achievable.
- 2) Combined “black” and “grey” water i.e. sewage drainage system which is the common practice, with separate storm water drainage
- 3) Combined “grey”, “black”, and storm water drainage systems. It is important to note that this type of system is rarely used in practice nowadays because of the extreme differences in flow that the treatment plants, pumping stations are subjected to, reducing their operating life. (Cotton and Tayler, 2000)

### **5.3.1 Combined “grey” and storm water drains with separate sewage water drains**

It is recommended that combined sullage and storm drains are able to carry the high flows resulting from intense rainfall and the very low flows of sullage during dry weather periods at velocities sufficiently high to prevent deposition of solids (Cotton and Tayler, 2000). The combined sewers should be able to carry the combined flow at the gradient selected of either the minimum allowable gradient or that the slope available to maintain minimum cover (i.e. ground slope or can be influenced by the minimum allowable cover). In this case the ground slope is selected.

The grey and storm water flows used in sizing drain are computed as follows:

#### **5.3.1.1 Grey water flows**

The grey water generated in Alexandra was investigated for only the residents occupying backyard shacks. It was estimated to be approximately 40% of the average wastewater flows generated for the three households monitored. This percentage of the total average dry weather flows derived from measurements taken at the station F1 for the sub catchment A and the southern culvert storm water drain is used in sizing the outfall sullage drain as follows:

$$\begin{aligned}\text{Sullage average flow rate, } S_a &= (0.4 \times 487) \\ &= 194.8 \text{ l/s}\end{aligned}$$

$$\text{Peak sullage flow rate, } S_p = \text{peak factor} \times S_a$$

The peak factor used is determined from the maximum average daily flow rates and the average daily flow rates measured for all the measurement locations, and it approximates to 1.5, even for the wet weather flow measurements taken in November by PH&R.

$S_p$  therefore = 292.2 l/s

Waste sewers in South Africa are designed to run full at the design flow and gradient, (DCD, 1983). In order for allowance to be made for possible increases in water consumption the pipe capacity is determined on the assumption that sewers are designed to run not more than half full at the design flow and gradient, i.e.  $y/D = 0.5$ , where  $y$  is the depth of flow and  $D$  is the diameter of pipe.

From the chart of partly full circular drains (see appendix C) the corresponding  $Q/Q_f$  ratio is 0.5

$$\begin{aligned}\therefore Q_f &= S_p/0.5 \\ &= 292.2/0.5 \\ &= 584.4 \text{ l/s}\end{aligned}$$

### **5.3.1.2 Storm water flows**

The storm water flows to be drained would be for the 10-yr level flood considering that the major floods are less frequent, and the combination of open spaces and roadways considered adequate to handle the runoff flows generated. The 10-yr level flood peak flows from each of the three south sub-catchments were computed using RAFLS.

The sum of the peak flows generated from each of the three south sub-catchments for the 10-yr RI and the peak flows computed for the grey water are used to size the outfall sewers by substituting into the manning equation, they are summarized in table below.

Table 5.1: Computed peak flows of each waste stream from the south section of study area

Grey water peak flows, l/s	Storm runoff peak flows, m <sup>3</sup> /s			Combined peak flows, l/s
	SS1	SS2	SS3	
584.4	23.74	4.24	16.72	45,284.4

The drain is designed to flow full and a manning coefficient of 0.015 for non-pressure reinforced concrete pipe material is used as recommended for storm water reticulation systems in South Africa. (SAICE, 1976)

The manning formula is used in pipe drain size determination and is given by:

$Q = (1000 * A R^{2/3} S^{1/2})/n$  where,

$Q$  = Capacity in liters/second, l/s

$A$  = Cross-sectional area of flow, m<sup>2</sup>

$S$  = Longitudinal slope (taken as the ground slope)

$$\begin{aligned}
N &= \text{manning roughness coefficient (to be taken as 0.013 for vitrified clay pipe material)} \\
R &= \text{the hydraulic radius represented by } A/P, \text{ where } P \text{ is wetted perimeter} \\
&= D/4 \text{ for circular cross-sections} \\
\therefore D &= \{(45,284.4 * 0.015 * 4^{5/3}) / (\pi * S^{1/2} * 1000)\}^{3/8} \\
D &= 2.25\text{m}
\end{aligned}$$

### **5.3.1.3 Black water flows**

The black water constitutes 58% of the total average daily wastewater generated from the sewer sub-catchments A as measured from station F1 and the southern storm culvert, and it is used to size drains as follows:

$$\begin{aligned}
BW_a, \text{ l/s} &= 0.58 * 487 \text{ l/s} \\
BW_p, \text{ l/s} &= 1.5 * 282.46 \\
&= 423.69 \text{ l/s}
\end{aligned}$$

Since the design adopted is for foul sewers running half full at the peak flow, the ratio of depth of flow to diameter,  $y/D$  is 0.5. From the chart showing hydraulic properties of partly full circular pipes reproduced in appendix C, the corresponding  $Q/Q_f$  is also 0.5. The value for  $Q_f$  is then computed as:

$$\begin{aligned}
Q_f &= BW_p / 0.5 \\
&= 423.69 / 0.5 \\
&= 847.38 \text{ l/s}
\end{aligned}$$

The pipe capacity required to carry this flow is computed using the manning equation above as follows:

$$\begin{aligned}
Q &= (1000 * A R^{2/3} S^{1/2}) / n \\
\therefore D &= \{(Q * n * 4^{5/3}) / (\pi * S^{1/2} * 1000)\}^{3/8} \\
D &= \{(847.38 * 0.013 * 4^{5/3}) / (\pi * S^{1/2} * 1000)\}^{3/8} \\
D &= 0.498 \text{ take 500mm the next adequate diameter size}
\end{aligned}$$

The outfall pipe drain costs for the combined system arrangement (1) are summarized in the table below. They are based on a theoretical rate of R 5,000 per meter diameter, per meter length of pipe.

Table 5.2: Pipe drain sizes, and costs for combined system arrangement (1)

	<b>PEAK FLOWS, m<sup>3</sup>/s</b>	<b>DIAMETER, mm</b>	<b>COST (per meter length), Rands</b>
Combined sullage and storm water drain	45.284	2,250	11,270
Separate “black” water drain	0.847	500	2,500
<b>Total cost</b>			<b>13,770</b>

### 5.3.2 Combined black and grey water drainage system with separate storm drainage

This arrangement is commonly used as grey water normally fed into sanitary sewers where they are installed. In this case the storm drain system designed to take the sum of the 10-yr flood peak flows from the south sub-catchments. The sewage flow used in sizing drains is the dry weather flow as recorded during measurements having applied the peak factor of 1.5 and also allowing for possible increases in water usage by designing for pipes that run half full rather than full at the peak design flow. The computations are as follows:

#### 5.3.2.1 Sewage flows

The total average daily wastewater generated from the sewer sub-catchments A as measured from station F1 and the southern storm culvert, are used to size drains as follows:

$$\begin{aligned}
 BW_a, \text{ l/s} &= 487 \text{ l/s} \\
 BW_p, \text{ l/s} &= 1.5 * 487 \\
 &= 730.5 \text{ l/s}
 \end{aligned}$$

Since the design adopted is for foul sewers running half full at the peak flow, the ratio of depth of flow to diameter, y/D is 0.5. From the chart showing hydraulic properties of partly full circular pipes reproduced in appendix C, the corresponding Q/Q<sub>f</sub> is also 0.5. The value for Q<sub>f</sub> is then computed as:

$$\begin{aligned}
 Q_f &= BW_p / 0.5 \\
 &= 730.5 / 0.5 \\
 &= 1,461 \text{ l/s}
 \end{aligned}$$

The pipe capacity required to carry this flow is computed using the manning equation above as follows:

$$\begin{aligned}
 Q &= (1000 * A R^{2/3} S^{1/2}) / n \\
 \therefore D &= \{(Q * n * 4^{5/3}) / (\pi * S^{1/2} * 1000)\}^{3/8} \\
 D &= \{(1,461 * 0.013 * 4^{5/3}) / (\pi * S^{1/2} * 1000)\}^{3/8}
 \end{aligned}$$

D = 0.611 take 700mm the next adequate diameter size

The costs in this table also computed basing on the theoretical rate of R 5,000 per meter diameter, per meter length.

Table 5.3: Pipe drain sizes and costs for combined system arrangement (2)

	<b>PEAK FLOWS, m<sup>3</sup>/s</b>	<b>OUTFALL PIPE SIZE, mm</b>	<b>COST, (per meter diameter per meter length) RANDS</b>
<b>Sewage outfall drain</b>	1.461	700	3,500
<b>Storm water drain</b>	44.7	2243	11,215
<b>Total cost</b>	<b>14,715</b>		

### 5.3.3 Combined system for all three waste streams

With this arrangement all three waste streams from the entire south sub catchment are drained through the same outfall drain. The drain capacity is determined for the sum of peak sewage flows and storm runoff peak flows for the 10-yr level flood on the south sub-catchments using the manning equation, the coefficient, n is taken to be 0.015.

Table 5.4 Pipe drain size and cost for combined system arrangement (3)

	<b>PEAK FLOW, m<sup>3</sup>/s</b>	<b>DRAIN SIZE, mm</b>	<b>COST, RANDS</b>
Combined “black”, “grey” and storm water outfall drain	46,161	2300	11,351
<b>Total cost</b>	<b>11,351</b>		

## 5.4 COMPARISON OF THE POSSIBLE DRAINAGE SYSTEM ARRANGEMENTS

Table 5.5: Summary of costs and other facts on the three possible system arrangements

SYSTEM TYPE	GENERAL REMARKS	COST, Rands
Combined system type (1)	<ul style="list-style-type: none"><li>• Separation at downstream end achievable</li><li>• Treatment of the concentrated stream requires simple methods like skimming, settling or screening</li></ul>	13,770
Combined system type (2)	<ul style="list-style-type: none"><li>• It is common for grey water to be fed into the sewers where sanitary sewers are installed</li></ul>	14,715
Combined system type (3)	<ul style="list-style-type: none"><li>• Not viable in areas with extreme climates where extreme or infrequent storms make separation difficult.</li><li>• Sanitary conditions in peri urban settlements could result in increased blockages</li></ul>	11,351

From table 5.5, it appears that in regards to cost a combined system for all three waste streams (i.e. combined system (3)), would be the most economical arrangement, however, the solid waste disposal practices and alternative cleaning methods characteristic of peri-urban settlement dwellers may result in operation and maintenance difficulties (Stephenson, 2001).

The combined system arrangement (1) is more economical than the combination of separate sewage and storm drains arrangement, which is the common practice. The combined system arrangement has the added advantage of ease in separation and treatment of the concentrated stream on the downstream end.

## CHAPTER SIX      CONCLUSIONS

The purpose of this research was to identify appropriate technology options for drainage of low-income peri urban settlements of South Africa, and Alexandra west bank was selected as a case study example for this study.

A review of literature on the subject of drainage reveals that drainage flows can either be managed on or off-site and table 3.1 summarizes the particular technology options falling under each approach. The selection of the appropriate approach and the associated technology options is dependent on several factors, however only compatibility with physical site conditions, drainage flow quantities generated and cost were used in screening of options for this research study. On site drainage technology options are largely dependent on the restrictions presented by the local site conditions, they are however, generally considered to be affordable. Off site drainage options on the other hand require significant financial investment, and thus cost is a key comparison factor.

### A.      PHYSICAL SITE CONDITIONS

Space availability, general topography, soils and geology are the physical site conditions that have a major bearing on the feasibility of on-site drainage options.

Congestion is a common characteristic of low-income peri-settlements in South Africa and it arises from high population densities and the haphazard establishment of informal dwelling structures in service and open space areas. In the study area, which is in a flood plain region, shacks are erected in the backyards of formal residential stands, along the tributaries of the Jukskei River, and above storm drainage, water supply and sewerage reticulation system pathways. The population density stands at over 1000 persons/hectare, and though de-densification is planned, reduction in densities will be by only about 10%.

Topography, soil and geology are more site-specific conditions, and thus description provided applicable to the study area, Alexandra West bank; gently rolling terrain with slopes ranging from 5% - 12.5% in the west-east direction; in-situ soils are of the sandy clay or clayey sand type as per the unified classified system, they together with the gully and hilly wash overlay average a depth of 9m to bedrock; the water table is perched in certain areas, but generally ranges from 1.3-7.8m in depth. Close proximity of water table level in certain



areas and the poor drainage capabilities of the in-situ soils in general, are a hindrance to approaches that involve exfiltration through soil.

Onsite drainage solutions essentially consist of storage or infiltration techniques that require substantial land area and/or free draining soils therefore intensity of development coupled with poorly draining soils restrict the use of on site management approaches in Alexandra west bank.

## B. DRAINAGE FLOWS

The quantity of drainage flows generated within the settlements is another factor that influences the choice of drainage option. The grey and black water flows are the result of predominantly daily household water use, while storm water runoff flows are intermittent being rainfall related. The quantities of flows generated in Alexandra West Bank Township were determined from field observations, flow measurements and hydrologic simulation.

Field observations of water use and wastewater disposal practices of three informal household type dwellers indicate that:

- Water consumption fairly constant throughout the day, and this is attributed to the large portion of the resident population that doesn't move out of the area to go to the workplace.
- Approximately 85-94% of the water consumed in the households is considered to be what would end up in sanitary sewers, (assuming no wastewater discharged to the streets or alleys), in addition to flows from GWI and SWI that are considered to constitute flows in these sewers. This value was determined on the assumption that apart from the water used for physiological needs (i.e. cooking and drinking) the balance ended up as wastewater.
- The total household wastewater flows generated constitute approximately 40% grey water and 58% black water flows, the remaining 2% being water running along streets and alleys from vehicle washing and that used in containing dust pollution.

These findings are assumed to apply even for the other household types (i.e. flats/apartments, and formal houses) because there is no evidence of significantly different lifestyles (e.g. gardening and/or recreational use of water) of occupants of these other household types.

Wastewater flows generated were also monitored at the outfall end of the study area, from four major locations: the outlets of storm culverts, and along the collector mains serving two of the largest sewer drainage areas. The findings show a discrepancy in the values of average daily per capita flows and wastewater production rates computed at the household level and

those derived from population estimates for the selected sewer drainage area and the wastewater flows measured at its outfall end. The average daily per capita wastewater flows stand at 35l/c/d, and 83l/c/d, at the household and outfall end respectively, while wastewater production rates are at approximately 37m<sup>3</sup>/ha/day and 87 m<sup>3</sup>/ha/day. The variance is close to two and half times and has been attributed to water wastage resulting from defective plumbing, since the observations at household level, didn't take into consideration leaking taps, toilets and other plumbing fixtures.

If the assumption is made that wastage is curbed, the wastewater flow component only is still in excess of the 10m<sup>3</sup>/ha/day production limit beyond which, use of onsite drainage technology options not recommended.

Storm runoff flows generated within study area were determined by simulation using RAFLS software for the 10-yr flood level for the major drainage sub-catchments as shown in figure 4.12 and a stand size area. The flow rates computed much larger than the total wastewater flows, making it impossible to meaningfully compare quantities generated with the grey and black water components of the wastewater flows.

In the case of Alexandra west bank, the study area, and settlements with similar characteristics in regards to the site conditions and the drainage flows generated, onsite drainage not applicable because of lack of adequate space, poorly draining soils and excessive flows due to high population densities that would exceed the capacity of even freely draining soils.

### C. OFF-SITE DRAINAGE OPTIONS

Congestion and high population densities, characteristics common to low-income peri-urban settlements in South Africa make the off-site drainage approach inevitable. This approach includes a range of options based on certain technological criteria that can be applied to different conveyance system arrangements. For purposes of determining the most economical arrangement, outfall pipe drain sizes were designed for basing on the conservative technological standards for both foul and storm drains for the following system arrangements:

Type 1: Combined grey and storm water drainage system, with separate black water drainage system.

- Type 2: Combined black and grey water i.e. sewage drainage with separate storm drains
- Type 3: Combined grey, black and storm water drainage system.

The type 2 and 3 arrangements are the traditional off-site drainage system arrangements, while type 1 is one of the newly emerging alternative conveyance system arrangements. Using a theoretical cost of R 5,000 per meter diameter, per meter length these three system arrangements were compared and it was found that the type 1 arrangement would be the more economical. This system arrangement, also allows for ease of separation between the liquid and solid portions, using simple methods like skimming and settling, which would contribute to reduced costs in regards to the quality aspect of drainage.

The theoretical cost comparison excludes costs for treatment, operation and maintenance and thus not conclusive in it self. The treatment of the separate and combined grey and storm water drainage streams has previously not been practiced and thus information on treatment as well as operation and maintenance costs not readily available. There is an ongoing study on treatment options for these streams when separate and/or combined but findings were not available for inclusion in this report.

The operation and maintenance costs for these three options, however not expected to change the cost comparison results significantly because, as mentioned in *section 3.3.3* of the literature review, type 1 arrangement limits community to on-site disposal for the black stream through VIP toilets in rural areas and chemical toilets for peri-urban areas. These on-site disposal options have less operation and maintenance requirements than off-site conveyance systems.

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## **APPENDICES**